POST CONSTRUCTION MONITORING OF ASH DYKES

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by
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ABSTRACT

Fly ash is a major industrial waste produced in India The low calorific value and high ash content of coal has resulted in enormous production of fly ash by various Thermal Power Plants in India whereas its utilization is very limited Therefore, it is generally disposed off in the vicinity of the Thermal Power Plants in the form of ash mound/dykes In the present investigation, post construction monitoring of ash dykes raised by upstream method of construction has been carried out at the ash disposal sites of Korba Super Thermal Power Plants (KSTPP), Korba and Badarpur Thermal Power Station (BTPS), New Delhi

In case of ash dykes, primary importance is that of the horizontal, vertical movements and pore water built up within the dyke and therefore, settlement plate, inclinometer and piezometers were installed at KSTPP ash dyke Standard Penetration test have also been performed at critical locations to study the strength properties of ash The results of inclinometer reveal that the lateral movement was confined to the ash filling operation of lagoons, while the results of settlement plate show that nearly 98-99% of the settlement of the lagoon ash occurs during the construction of ash embankment over it, assuming double drainage of the compressible lagoon ash layer beneath the embankment In case of single drainage percentage of settlement depends on the depth of compressible layer The total settlement is found to be 3 -4 % of total depth of compressible layer of ash Piezometers are necessary to check the phreatic line, on which the stability of dykes are dependent It also assures the proper functioning of drainage blanket provided within the embankments.

SPT, piezocone and plate load tests were performed at BTPS ash dyke site to evaluate the improvement in shear strength property of settled ash due to the construction of ash embankment over the settled ash These tests exhibited an improvement in the angle of internal friction ϕ of lagoon ash from 28° to 31°. The same degree of improvement was found for settled ash at KSTPP ash dyke. The meager improvement of ϕ by 3° has greatly enhanced the chances of further raising at KSTPP ash dyke A detailed stability analysis performed on ash dykes for each seismic zone of India assuming same degree of improvement in the shear strength of ash has revealed that eight raisings are possible in low seismic zones In high seismic zone only three raisings are possible In a very high seismic zone (Zone V), ash embankment should not be constructed by upstream method of construction

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Chapter 1

Introduction

Considering the size of its economy, India is poorly endowed with energy resources. It's major source of commercial energy is coal. Current reserves are sufficiently large to meet India's coal demand over the next 250 years By comparison, oil and gas reserves will last only for another 15 - 20 years. Coal currently meets about 2/3 of India's total energy needs. With the commissioning of super thermal power plants and increasingly use of low grade coals of high ash content the current production of ash is about 60 million tons per year This figure is likely to go upto 100 million tons per year by the end of this century. The map exhibiting thermal power plants in India have been shown in Fig. 1 1 .

The power generation has recorded a 42 fold increase between 1950- 51 and 1991- 92, however, the availability of energy is significantly short of demand Since electrical energy plays a crucial role in both industrial, commercial and agricultural sectors and thus the consumption of electricity in the country has been used as an indicator of productivity and growth. With this perspective, power development has been given high priority in the development programme of our country

The total energy requirement of India at the end of the century is estimated to be around 1,10,000 MW as against around 85,000 MW at present Coal consumption by power sector was 46 % of the total production in 1985-86, which has increased to 60 5 % in 1992 -93 and is likely to reach 74 % of the total production by 2001-02. Table 1.1 shows the coal consumption by various Industrial sectors of India

Coal fired plants generate different types of ash residues and discharge enormous quantities of particulate material & vapours. Fly ash is the most abundant of all the residues Flyash from power plants is posing a major environmental problem to power plant authorities Its disposal not only requires huge land, water and power, but also causes ecological problems.

Different types of coal produce different quantities of ash, depending on the concentration of mineral matter in that type of coal In the western countries, the coal is generally processed prior to delivery at the power plant and therefore the ash content is generally less than 20 % of the coal. In India the coal contains very high percentage of rock and soil and therefore the ash contents are as high as 50%

At present only a negligible percentage of fly ash is utilized Recent researches has shown that its application potential is tremendous not only in the field of civil construction but in the industrial & agricultural applications also Some of the uses of flyash are in construction of road embankment , minefields, lime fly ash concrete, flyash bricks, Portland pozzolana cement, light weight aggregates etc Since, Indian fly ash generally consists of silica, alumina, hematite, unburned carbon & small amount of alkalis and sulfates, therefore it could be used in extraction of metals from them. But these usage of flyash are very less as compared to its production. Until full utilization of ash is made, ash produced from the power plants will be continued to be disposed off on ground as a waste material Although ash dyke is a tail end of the Thermal Power Plants (TPPs) , even then disruption in the ash disposal may hamper the production of electricity Since the volume of ash to be disposed off is large, it needs proper ash management programme It can be disposed off either by dry scheme or by wet scheme

Until 100 percent utilization of ash is made the lagoons will keep on filling When the lagoons become full, four basic options are available : (a) constructing new lagoon using either conventional construction material, (b) having the flyash in existing lagoons hauled to another disposal site, (c) raising the dyke using flyash excavated from the lagoon. The fourth option is considered to be very cost effective because any flyash used for constructing dyke would in addition to saving the earth filling cost, enhance the disposal capacity of lagoon.

In India, ash from the most of the power plants is disposed off by wet scheme in the vicinity of the power plants by constructing ash lagoon(s). At several sites where the lagoons become full the fourth has been implemented.

Currently at several sites e.g. at Korba, Delhi, Bokaro, Singrauli, Vindhyachal, Ramagundam, Farakka etc existing lagoons have been raised by upstream method of construction. In this case ash embankment is raised over the settled ash. The design has been developed based on conservative assumption The upstream method of raising the embankment requires the highest level of monitoring as the embankment rests more and more on loose deposits as the embankment height increases.

The present thesis covers the post construction monitoring of flyash dykes raised by upstream method of construction. This includes the measurement of the geotechnical properties of the ash prior to construction, during construction and post construction and to measure the improvement in the shear strength of ash due to consolidation either due to time or due to overburden load of the embankment. The improvement may allow us for further raising of the ash dyke safely. The monitoring of post construction performance of ash dykes such as vertical, horizontal movements and pore water pressure measurements reveals the behaviour of ash dyke after construction and provides a check over the initial design made in the construction of ash dykes.

Following two sites were selected for detailed monitoring

- Korba Super Thermal Power Plant (KSTPP), Korba
- Badarpur Thermal Power Station (BTPS), New Delhi

In chapter 2 , the properties of ash, types of disposal system, methods of evaluating shear strength using field tests such piezocone, standard penetration test and plate load test have been reviewed

Chapter 3 deals with the various instruments used during monitoring of ash dykes at the selected sites. The horizontal and vertical movements of the dyke were measured through inclinometers and settlement plates respectively The water pressure developed within the body of the dyke was monitored through piezometers.

Chapter 4 deals with the various field and laboratory tests conducted at the selected sites The test results are also discussed in this chapter

Chapter 5 deals with the correlation of results obtained through various field and laboratory tests. This chapter also studies the impact of the results obtained, on the stability of the ash dyke.

Chapter 6 is the summary and conclusion of the present investigation.

Table1.1 : Coal Consumption : Industrial Sector, India

(million tons)

Sector	1976-77	1978-79	1983-84	1988-89
Power	27.0	33.7	55.2	87.3
Steel	22.3	24.6	28.7	32.4
Railway	13.3	13.5	12.0	10.0
Domestic	7.7	8.5	10.4	10.8
Other	28.8	33.6	46.4	56.7
Total	99.1	113.9	152.7	197.2

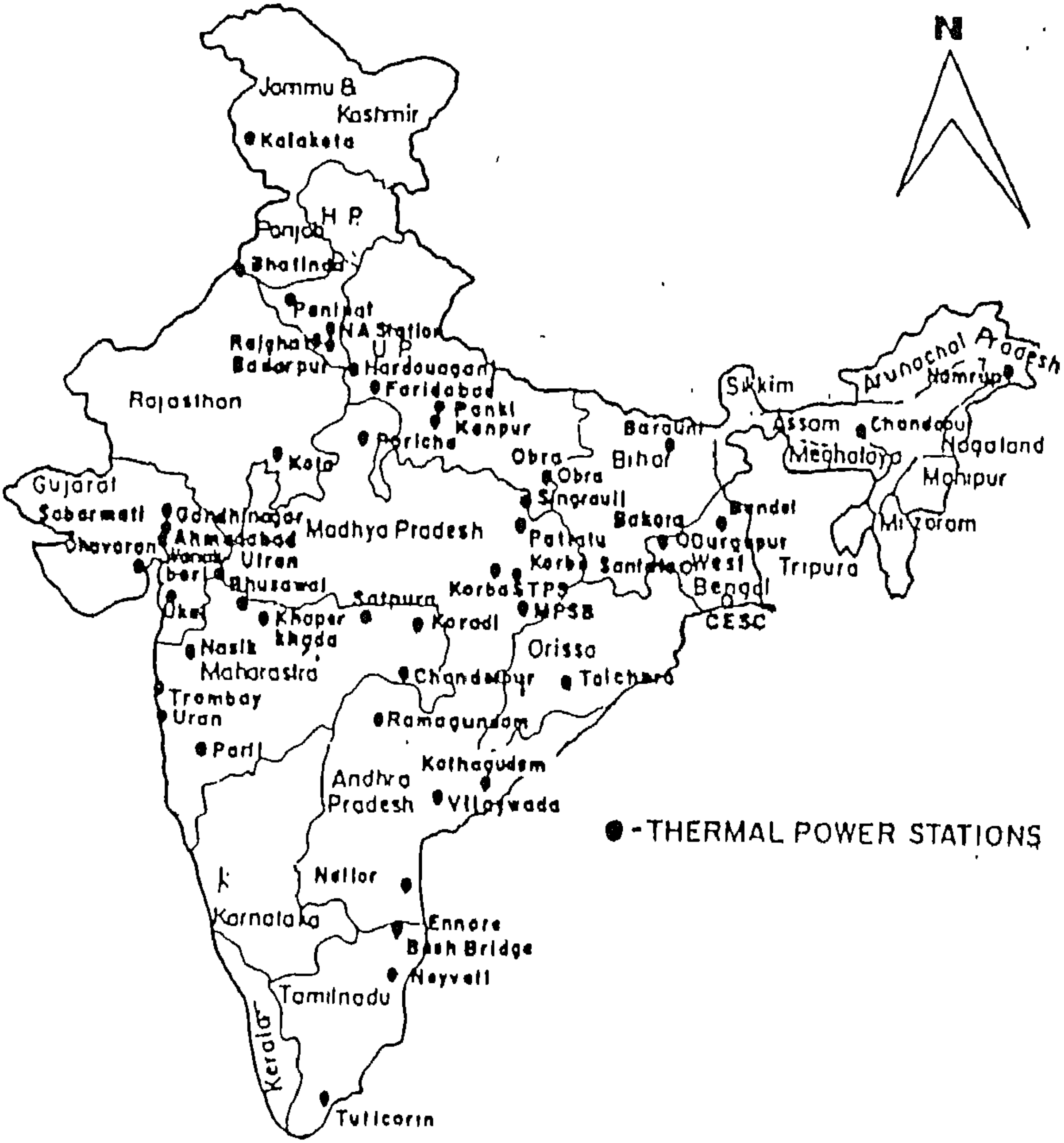


Fig.1.1 Thermal power plants in India

Chapter 2

Literature Review

2.1 General

A literature survey, concerning with the properties of fly ash and various types of disposal systems of ash being practised in India, have been conducted in this chapter The deffetent methods used to evaluate the shear strength of the cohesionless soil from the test results of static cone penetration test, plate load test and standard penetration test are also discussed in this chapter. The pollution caused due to ash dykes and their remedial measures have also been included within the scope of present study.

2.2 Geotechnical Properties of Fly Ash

Physico-chemical properties of fly ash need to be considered prior to any meaningful evaluation of possible utilization options or safe, cost effective and environmentally acceptable disposal methods The chemical and physical composition and the production of a fly ash is a function of several variables such as

i) coal source,
ii) Degree of pulverization,
iii) Design of boiler unit,
iv) Loading and firing conditions ,
v) Handling and storage methods.

Thus, it is not surprising that a higher degree of variation can occur in fly ashes, not only between power plants but within a single power plant also. A change in any of the above factors can result in detectable changes in the fly ash produced. The degree to which any change affects the utilization potential of fly ash is a function of the nature and degree of the change and the particular application for which the fly ash might be used

2.2.1 Types of Fly Ash

Fly ash may be classified into two groups, based on the nature of their chemical constituents .

i) bituminous- type or Class F flyash
ii) lignite- type or Class C flyash .

The bituminous fly ash is produced by burning bituminous or anthracite coal, and the Lignite fly ash by burning sub-bituminous coal or lignite.

Lignite type ash is defined as an ash having more calcium oxide (CaO), plus magnesium oxide (MgO), than ferric oxide (Fe_2O_3) By contrast , the bituminous -type ash will have more ferric oxide, than calcium oxide (CaO), plus magnesium oxide (MgO) Geologically, bituminous ash is produced from the Triassic coals and older coals and lignite type from Jurassic and younger coals. The average particle size of lignite fly ash is considerably coarser than that of Bituminous fly ash Also, free lime is present in all lignite type fly ash and therefore it has self cementing property.

Bituminous fly ash is the most common It has pozzolanic properties but is not appreciably self cementing Because of the geographic distribution of coal types, Bituminous fly ash is principally produced in India. Although less common , the Lignite flyash is produced in South India in the vicinity of Neyvelli

2.2.2 Chemical Properties of Ash

Fly ash is a silt-sized, fine grained material In an aqueous environment, the ash reacts with lime (CaO) and produces a cementitious material with good structural properties This is termed as pozzolanic property and it form the basis of many uses of the ash The amount of CaO is very much influenced by the coal sources while the carbon content is function of efficiency of the particular boiler unit and the fineness of the pulverized coal. The free lime influences the pozzolanic activity of the fly ash, while unburned carbon affects the compaction and strength characteristics. The properties of fly ash, however, vary over a wide range depending upon the type of coal, furnace and burning temperature and other factors. For example, the ash resulting from combustion of bituminous coal or anthracite shows no significant self cementing properties.

The four principal groups of mineral found in India Coals are hydrated silicate group (kaolinite); carbonate group (Calcite and sederite), sulphate group (feldspar, quartz, apatite, etc)(Torrey 1978).

The constituents most likely to affect the index and engineering properties of ash are silica, iron, free lime and carbon The ashes containing higher percentage of iron content will have higher specific gravity as compared to ash conataing higher silica content. The chemical composition of ash influncences its colour to a large degree Ashes range in colour from cream to dark brown or gray.

The cream colour is usually produced by a high calcium oxide content, and gray to black by increasing quantities of carbon Table 2 1 gives the range of chemical composition of Indian fly ashes along with the average values of the US, UK and Swedish ash

2.2.3 Specific Gravity

Specific gravity is frequently needed in analyzing ash for evaluation of many geotechnical properties Fly ash is characterized by low specific gravity, uniform gradation and lack of plasticity . The specific gravity of fly ash particles depends upon chemical composition and generally the specific gravity of most of the ashes are considerably less than that for soils Table 2 2 gives the range of specific gravity of bottom ash, fly ash and pond ash in India and other countries It is observed from the table that the specific gravity of the bottom ash is slightly higher than fly ash which in turn is higher than that for pond ash. One of the reasons for the low specific gravity of fly ash as compared to soil is because of the high proportions of ash particles being cenospheres or hollow particles. The large variation in the values of specific gravity is because of the combination of many factors such as gradation, particles shape, composition and segregation (Gray and Yen-Kuang Lin, 1972). The degree of segregation depends on the material distance from the discharge point. The specific gravity of the ash particles at the discharge zone is higher than that at the outflow point. Also, the specific gravity of the ash is higher when it has high iron content.

Generally the specific gravity of the Indian ash vary from 1.7 to 2.4. The mean value is 2.1 . The American Class- F ash exhibits a mean specific gravity of 2.4 while that of Class- C ash shows a value of 2.5

2.2.4 Grain Size distribution

The grain size distribution of ash provides considerable insight into its properties and behaviour. Fly ash is a typically uniform silt sized material. Figure 2.1 presents a typical grain size distribution curve for flyash, bottom ash and pond ash It is observed that the bottom ash and pond ash are coarser than fly ash (Sridharan et al 1995). The average particle size of lignite flyash is found to be considerably coarser than the bituminous fly ash.

Table 2.3 shows the range of values for particle size of some Indian ash (Rajasekhar et al, 1995). It is observed from the table that grain size at the 10th percentile, d_{10}, which influences the hydraulic behaviour (permeability and capillary moisture retention) of non-plastic soils ranges from 0.0027-0 02 mm for fly ash where as for bottom ash it is from 0 08 mm to 0 7 mm. The range of C_u (coefficient of uniformity, defined as the ratio of d_{60} over d_{10}) for fly ash is from 13 3 to 10.0 while for pond ash is 7.1 to 5 3. The coefficient of curvature varies from 2 16 to 0 73 for bottom ash, from 1.2 to 0.95 for pond ash and 0 83 to 1.03 for fly ash. The mean coefficient of uniformity of the US ash is 5.94 . It is to be noted that Indian ashes are relatively coarser than the US ashes

The fly ash is typically a uniform silt size material and the grain size of bottom ash varies between sand to fine gravel. Because of spherical shape, small surface area, uniform size of the particles, most of the ashes have no plasticity The gradation and fineness are greatly influenced by the degree of pulverization system and the efficiency of precipitator system.

2.2.5 Atterberg Limit

Atterberg limits include liquid limit, plastic limit and shrinkage limit. The values of liquid limit and plastic limit are very useful in the classification of soils. They also provide an overall idea for the engineering properties of the soils with liquid limit. Since, the ash is non plastic, it is very difficult to conduct a liquid limit test using Casagrande method, but one can get the liquid limit value using cone penetration method. Since, fly ash is not plastic in nature, plastic limit cannot be determined. The liquid limit of the fly ash is found to be in range of 43 to 49% (Rajasekhar, 1995) It's shrinkage limit varies with initial water content The liquid limit of pond ash tends to be much less than that for fly ash.

Spherical shape, small surface area, and uniform silt to sand size of individual particles make ash non-plastic

2.2.6 Compaction Characteristics

The density of ash is an important parameter since it control its strength, compressibility and permeability Compaction is a process of increasing the density of the material by packing the particles closer together with reduction in the volume of air. Densification of soil/ash improves the engineering properties. The degree of compaction of soil/ash is measured in terms of dry density The compacted unit weight depend upon the amount and method of energy application, and material properties such as grain size, gradation, particle shape, plasticity and the moisture content at compaction Fly ash is non plastic and uniformly graded indicating the need for vibratory densification, but the fineness of the particles also suggests that high pressure might be necessary Fly ash becomes unmanageable when it is either dry or saturated However, a well compacted flyash exhibits strengths comparable to those for soils normally used in earth fill operation. Fly ash compaction might thus be expected to be a hybrid experience with fine and course grained soils..

Normally , the dry density of the pond ash when tested under standard proctor compactive effort ranges between 0.91 to 1.49 g/cc and the optimum moisture content varies from 18 to 45 % (Skarzynska et al. 1989). It can be also noted that the maximum dry density (MDD) and optimum moisture content (OMC) of the same pond ash but collected from different places (i.e., ash at slurry discharge point and at water escape) could be different (Dayal et al,1988). The pond ash collected near the ash slurry discharge zone gives higher dry density and lower OMC values than for the ash collected near the water escape point This is because of the grain size distribution aided by the specific gravity of the particles at the slurry discharge zone being higher than that of the particles near the water escape point.

The mean value of maximum Standard Proctor Dry Density and the Optimum Moisture Content of Indian fly ashes, bottom ashes and combined ashes are given in the Table 2.4 (Dayal ,1988)

It is clear from the above table that the higher values of MDDs are related to lower values of OMC's The Maximum Dry Density and Optimum Moisture Content increase with the increase in the specific gravity This fact is clear from the Figure 2 2 showing curves of MDD with OMC at different specific gravities For bottom ash MDD -OMC curve is almost a straight line which shows that the moisture content does not significantly affects the compacted density

2.2.7 Coefficient of Permeability

The bottom ash and pond ash are more permeable than fly ash This is understandable because the bottom ash and pond ash being coarse grained and devoid of fines compared to fly ash and thus have a higher value for permeability coefficient For comparison purpose the coefficient of permeability values for some other fly ashes, bottom ashes as well as for soils are also given in the Table 2.5. The value of coefficient of permeability also depends on the degree of compaction and pozzolanic activity The cementation causes a reduction in permeability It has been reported that for the values of MDD varying from 1.4 to 1 23 gm/cc the corresponding values of coefficient of permeability vary from 1.65×10^{-5} to 1.34×10^{-4} cm/sec. In case of KSTPP ashes, for the range of MDD from 1 21 to 1.01 gm/cc the corresponding values of permeability coefficients vary from $2\ 25\times 10^{-5}$ to 1.34×10^{-4} cm/sec Generally the permeability of Indian ashes are higher compared to the US. this may be attributed due to presence of lesser amount of finer particles in Indian power plant ashes. The in situ horizontal permeability of the pond ash is normally 10 to 15 times greater than insitu vertical permeability (Sridharan, 1995) This is because lagoon deposits are highly stratified. Some of the fly ashes do have permeability values as low as those for clayey soils This aspect is very useful in utilization of ash as a low permeable barrier to reduce the migration of solutes and a possible ground water pollution.

The use of low permeability ash as a fill produces a structure with high run-off factor and it may be necessary to prevent the external erosion effects.

It has been found that the coefficient of permeability of fly ash decreases with increasing compaction.

2.2.8 Consolidation and Compressibility Characteristics

To estimate the settlement of fly ash embankment or fills, one-dimensional consolidation test results are required. Results of laboratory consolidation tests (Capco, 1990) on undisturbed samples from reclamation fills gave values of coefficient of consolidation, C_v of the order of 0.57 -1.14 cm²/min.(γ_d in the range of 0.75 to 1.3 g/cm³). But the actual in-situ values are much higher (around 3 to 10 times the laboratory values.) Since, primary consolidation for fly ash will be completed by about one minute it is difficult to take the time compression readings manually for C_v calculations using classical deformations *Vs* time plots. Webb and Hughes (1987) used an automatic recording system for taking the consolidation readings at very small time intervals. They have reported C_v values of the order of 1.9 to 19 cm² /min. These high rates of consolidation suggest that in most cases primary consolidation will be practically complete when the fill construction is over. This high rates of consolidation of fly ash is favourable particularly for its use as embankment and reclamation fills.

Equally important is the secondary consolidation which account for 20 -30 % of the total compression which depends on the nature of the fly ash and the duration of curing. The coefficient of secondary consolidation values decrease with curing time due to age hardening of fly ash. They are also highly dependent on the pozzolanic activity of fly ash which depends on the free lime content, carbon content and amount an chemical composition of the glass phase.

2.2.9 Shear Strength Characteristics

A number of ash characteristics effect design considerations and determine the suitability of a particular ash form dyke and embankment construction The most important among them are strength and permeability which determine its suitability for embankment construction.

Both total strength parameters and effective strength parameters are required in designs For example, in the case of dyke designs, total strength parameters are required for stability analysis during construction period or for sudden draw-down conditions whereas the effective strength parameters are required for designing the slope for steady seepage case and for the long term stability Strength properties of the fly ashes get affected by variations in density, moisture content, particle size distribution and chemical composition. It is important to note that the fly ash derives most of its shear strength from the angle of internal friction and that the cohesive strength parameters (c) indicates an apparent cohesion in the fly ash due to capillary forces produced by pore water This apparent cohesion can be destroyed by complete drying or saturation and , therefore must be ignored in the design of embankment or dyke (Mclaren and Digioa, 1987).

It has shown in the literature by Meyers, et al (1976) that the shear strength is affected by sample density and moisture content. The undrained shear strength decreases significantly in fly ash samples compacted on the wet side of optimum. In addition, shear strength decreases in samples compacted to less than maximum dry density. This is an important consideration in design since it is not often practical to require 100% compaction in the field. Normally a minimum of 95 - 90 % compaction is specified. For design purpose, shear strength parameters should be established at field compaction density.

The shear strength test results indicate that both effective and total angle of internal friction decreases as the density decreases (Fig 2.3) For 100 % Proctor dry density the average effective angle (ϕ') for combined ash is 36° and for 95% and 90% of Proctor dry density the average ϕ' is 33° and 30.5°, respectively (Dayal, 1988)

2.3 Types of Disposal

The ash generated in power plants is indeed, very huge in quantity Therefore the disposal of the same is a mammoth problem posed to thermal power station The general practice is of disposing it on isolated barren/waste lands, preferably low-lying But the identification and availability of such vast stretches of land is major problem, especially in a thickly populated country like India. There are two methods generally used in disposal of flyash namely, wet disposal and dry disposal

2.3.1 Dry Ash Disposal Systems (Ash Mound)

The dry ash storage mound is of dry ash placement type formed by using mechanical equipment to place the ash to pre-determined levels. A compacted earth toe embankment is constructed at the base of the disposal site with a compacted fly ash embankment face assuring a stable and durable disposal site. The size of the earth toe embankment varies and is dependent upon the magnitude of stabilizing force required to provide an adequate safety factor for the completed earth-fly ash embankment. The initial stage fly ash embankment consists of a compacted fly ash face portion and an up-stream uncompacted portion The fly ash within the face portion of this stage is placed in thin shifts and compacted to a dry density equal to 90 to 95 percent of the maximum dry density. The slope of the this initial stage of fly ash embankment depends on the strength properties of the ash being stored. Normally for a well compacted ash, a slope of two to three horizontal to one vertical is adequate. The remainder of the ash can be loosely placed or compacted for maximum capacity. This decision is based on economics and the planning of future use

The dry ash system is considered useful from the point of view of ash utilization The fly ash from ash mound is more in demand as compared to pond ash for the end use.

Drainage System. At a minimum, a blanket of bottom ash/granular fill is placed beneath all of the Stage I embankment This blanket collects any surface water which enters the site and percolates through the fly ash. The collected water is drained towards the bottom where it could be collected at predetermined intervals and conducted into the culvert A bottom ash drainage blanket is eventually placed under the entire disposal area The purpose of this extended blanket is to prevent a rise in the level of the groundwater table from inundating the fly ash, thereby eliminating leachate from this source. Thus, the extent of the bottom ash blanket depends on the ground water hydrology of each site.

Surface drainage is provided by a system of peripheral cutoff ditches constructed outside of the fill area to divert overland flow away from the fill area These cutoff ditches minimize erosion effects and reduce the volume of water that must be handled by the sedimentation pond which must be constructed downstream of the earth toe embankment. The fly ash embankment is shaped during construction to control runoff. Runoff from the fly ash embankment is collected in ditches and diverted to sedimentation ponds to provide siltation control. Comprehensive erosion and sedimentation control plans are a must for all disposal sites

All compacted surfaces of the disposal site are covered with soil and then seeded with a mixture of grass and legumes.

2.3.2 Wet Disposal

In this system ash is disposed in the form of 'ash slurry' by mixing it with water in the ration of 1:10 and pumping it through pipelines into ash pond. In such cases the area is raised by bunds all around the perimeter known as ash-dyke which is generally at an elevation of 8 to 10 m from original level and serve as a storage area.

Lagoon formation .- The entire ash disposal area is further sub divided into smaller areas by creating partition within the total area meant for disposal The lagoon formation helps in compartmentalization of the whole area and ash is disposed alternatively in each lagoon upto pre-determined levels before switching over to other lagoon

The advantage of lagooning is that while in one part of the dyke area where the ash disposal is continued, simultaneously from the other part reclamation of ash for any other useful purpose can be done But the main advantage of lagooning is that while the first lagoon is being discharged with ash the second can be raised during this period

Ponding of Ash : The ponding of ash is done with the set of ash disposal pipes through which the ash in slurry-form is discharged after pumping from ash pump situated within the plant boundary. The ash pipe line is placed all along the periphery of each lagoon with discharge end staggered throughout the perimeter of entire area, in order to facilitate ponding of ash uniformly.

Decantation : The decanted water as well as the rain water collected within the pond is disposed out of the ash pond through water escape structures. These structures should be located on the upstream side and beyond the toe of the ultimate phase dyke, otherwise, these structures has to built at each stage of dyke construction and connected to the water escape structures of previous stage. These structures are of such a height that it allows the water to spill over it's periphery or throught the adjustable precast slabs inserted in grooves of water escape tower The solid ash tends to settle at the bottom of lagoon and allows the water to collect at the top of ash bed, forming a water-cover. Sometimes the excess water is allowed to drain through a decantation toe having rock-filters to arrest the flowing of any excess ash The discharge water level is kept at the top of formation level where water is collected and gradually raised incrementally to subsequent levels Draw off pipes are also provide which are embedded below the earthen dyke/ash deposit. These pipes are encase in concrete to avoid leakage at joints. The discharge water from all lagoons is collected to a main drain which finally leads to surface water system

Toe-drain At the bottom of ash dyke throughout the perimeter of ash pond area a small toe-drain is made, which collects the seepage water through the ash-dyke and finally discharges into river/nallah

Land Requirement : The requirement of land for merely dumping/storage of coal ash generated by Thermal Power Plants runs in to several thousands of acres. According to one estimate 28,300 Hectares of land would be required for the storage of coal ash expected to be produced by several thermal power plants in India, by the year 2000 This is based on the assumption that the current rate of utilization will continue unchanged at 3 %. As a thumb rule 0 8 acres of land for the storage of ash is required for 1 MW of capacity of TPP taking average height of ash dyke as 8-10 m and total useful plant life as 25 years.

In India, ash from the most of the power plants is disposed off by wet scheme in the vicinity of the power plants as a waste material. Constructional methods for ash inpoundments can be grouped into three broad categories :

- Upstream method of stage constuction
- Downstream method of stage construction
- Centreline method of stage constuction

2.3.2.1 Upstream method

Fig. 2.4 shows typical embankment configuration constructed using the upstream method Similar constructions using other types of mine wastes have been practiced for many years and have been discussed by many investigators and designers. In this method an initial starter dyke is constructed at the downstream toe of the proposed embankment and then the embankment is reused The major advantages of this technique are that :

- It requires minimal volume of structural fill as part of new embankment is constructed on top of existing embankment and partly over the unconsolidated slurry already deposited,
- It provides better environmental pollution control compared to other methods because the constructed embankment being the final face of the ultimate embankment, vegetation and other fugitive dust control and/or leachate control measures can be planned on a permanent basis,
- Operational requirements such as haul and access roads, culverts, diversion and perimeter ditches may be constructed to serve the entire useful life of the facility,
- The watershed area decreases with the progress of embankment construction, requiring less volume for storm runoff handling with time,
- The starter dam, if proprly designed, can be used as a toe filter for the entire embankment.

The major criticism of upstream construction is that a part of embankment is constructed on the ash, which is not sructurally sound material However, by following the basic priciples of embankment design and through the judicious handling of materials, a safe dam can be constructed by the upstream method of construction. Although settled ash is moderately compressible, consolidation occurs in a relatively shot time period, thereby its implictions are normal, if the impoundment is located in a low seismic zone.

2.3.2.2 Downstream method :

In this method, construction is carried out on the downstream side of the starter embankment so that the crest of the dam is shifted progressively towards the downstream and the starter dam forms the upstream toe of the final dam The main advantages of this method is that none of the embankment is constructed on the uncompacted ash as in case of the upstream method and the embankment can be raised above its original ultimate design height without any serious limitaions and design modifications The placement and compaction control can be exercised as required over the entire fill operation Major drawback of this technique is that it requires relatively large volume of fill to raise the embankment The final outside face of the embankment is not created until the construction of last stage of disposal,which typically varies from 15 to 25 years.

2.3.2.3 Centerline method

It is basically a type of downstream method in which crest of the embankment is not shifted in the downstream direction but raised vertically upward above the crest of the starter dam This method has many design construction, environmental and operational problems and as such it is generally not adopted. The other drawbacks of this method are same as that of the downstream method of construction

A detailed comprehensive operational plan for disposal sited must be developed as a part of the construction package Since the filling operation of a disposal site extends for many years, engineering and environmental controls are extremely difficult to maintain unless a very specific plan exists In addition, with a detailed operation plan available, the disposal site can be more easily modified to conform to changes in disposal regulations.

2.4 In Situ Shear Strength Testing Methods

Although there are several direct and indirect methods for evaluating the in-situ shear strength of soil such as SPT, CPT, plate load test, vane shear test, pressuremeter test and dilatometer test, but only Piezocone (CPT), Standard Penetration Test and Plate Load Test are discussed here as these were the tests conducted at the two chosen sites.

2.4.1 Piezocone (CPT)

Piezocone is a most latest version of static cone penetration test equipment and is one of the most reliable test of measuring in situ shear strength of soil. Piezocone has a cross-sectional area of 10 cm² and an apex angle of 60° . The friction sleeve, located above the conical tip has the same diameter as that of cone tip. The cone is pushed into the ground with a constant speed of 2.0 cm²/min. The cone resistance (qc) and sleeve friction (fs) is measured independently and continuously with the penetration by means of load cell fixed inside the body of the instrument Piezocone also measures the pore water pressure developed during penetration With piezocone a pore pressure dissipation test can also be performed easily at the required depth At the depth at which a dissipation test is needed, the penetration is halted and measurements are taken

During the last decades several theories for calculating shear strength parameters have been presented from cpt tests In addition, purely empirical correlations based on laboratory tests conducted under controlled conditions have been suggested for estimating shear strength parameters and identifications of soil type. Several of these theories are summarized below for calculating angle of friction (ϕ) of cohesionless soil.

2.4.1.1 Angle of Internal Friction (ϕ') in cohesionless soils

De Beer Method (1945)

De Beer's old' theory is based on the assumption of an incompressible material and that cohesion can be neglected. This method originates from bearing capacity theory. This theory has been used widely, although it is resognised by the author (De Beer, 1974) that it gives very conservative value of ϕ'

Meyerhof Methods (1961,1974)

Meyerhof (1961) has presented bearing capacity factors for various angles of internal friction for rough and smooth cones and wedges of different apex angles as shown in Figure 2.5 correlation between limiting static cone resistance (The maximum value of q_c obtained at a critical depth, below which the penetration resistance shows little or no increase with continued penetration). Mitchel and Lunne (1978), found that this method gives ϕ' - values reasonably well where q_c does not increase significantly beneath a certain depth i.e. in case where critical depth is found

Muhs and Weiss Methods (1971)

Based on large scale model footings on sand Mush and Weiss (1971) have reported the following relationship,

$$q_c = 0.8\, N_\gamma$$

where q_c is unit cone resistance in kg/cm^2 and N_γ is ordianry bearing capacity factor. Because, N_γ depends only on ϕ', one can compute ϕ' indirectly. The result generally applies to the bearing capacity computation of shallow footings.

Janbu and Senneset Method (1973,1974)

Janbu and Sennset have proposed a method for determining ϕ' (effective) and, a (attraction) in sand and cohesive soils, which is based on bearing capacity theory modified by empirical observations. This thoery applies to situations where q_c profile increases linearly with depth in deposits assumed to have an approximately constant ϕ, and C over the depth interval (Fig. 2.6). The theory can best be described by a straight line given by

$\tau_f = (a + \sigma') \tan \phi'$

in which ,

τ_f = shear strength

a = 'attraction' ($a = \dfrac{C}{\tan \phi''}$) ,

σ' = the effective normal stress and

ϕ' = effective angle of internal friction

The following formula is given for cone resistance,

$$q_c + a = N_q\,(p' + a)$$

$$q_p = N_p\,(p' + a)$$

in which

$N_p = N_q - 1$

$q_p = q_c - p'$ = net cone resistance

N_q = bearing capacity factor, depending on tan ϕ'.

Hence, by plotting q_p versus effective overburden p′, one arrives at the following procedure of interpretation .

(I) Draw an average line through the variation of q_p with p′, The (negative) intercept on the p′ axis equals attraction a , beacause $q_p = 0$ when $p' = -a$

(ii) The slope of this line equals $N_p = N_q - 1$, Hence $N_p = N_q - 1$ yields tan ϕ' Thus the value of ϕ' can be calculated from tan ϕ' value

Janbu and Senneset have given values of tan ϕ' and *a*, in the following table, for the purpose of illustrating the order of magnitude obtained in the region of Norway.

Typical Values of Stength Parameters by Janbu and Senneset (1974)

Soil Condition	Friction tan ϕ'	Attraction *a*, KN/m²
Sand	0.65-0.95	0.900
Silt	0.50-0 70	0 300
Clay	0.35-0.60	0.120

Mitchell and Lunne (1978), applied this theory for determining ϕ' for number of tests sites and found that this method compare reasonably with actual measurements They confirmed that if a straight line portion of the q_c versus depth cannot be defined, Janbu and Senneset theory can not be used

Trofimenkov Method (1974)

A less conservative procedure than that of De Beer, with a semi-empirical basis , being used in USSR. A chart for determining ϕ from overburden pressure and cone resistance presents this correlation, and is valid up to an effective overburden pressure of 1 Kg/cm² This chart is shown in Figure 2 7

Durgunoglu and Mitchell method (1973, 1975)

This theory is based upon the results of laboratory tests. A rigid plastic, wedge-displacement bearing capacity theory was used, with empirical modifications to take into account the circular shape of the cone The ultimate static cone penetration resistance is given as

$$\frac{q_f}{\gamma_s B} = (\frac{C}{\gamma_s B}) N_c \xi_c + N_{\gamma q} \xi_{\gamma q}$$

For static penetration tests performed with a given cone there are many combinations of C and ϕ' which satisfy the above equation for a given value of q_f/γ_s If penetration data are available for two sizes of cone, or if the soil deposit is homogeneous and the penetration resistance is known at two depths, then specific values of C and ϕ' may be determinedd by simultaneous solution of two equations of the form of Equation , one for each combination of q_f and D/B values. Generally it is easier to eliminate C from the two equations and determine ϕ' by trial. For cohesioless soil the theory leads to

$$q_c = \gamma_s B N_{\gamma q} \xi_{\gamma q}$$

in which q_c = static cone point resistance

γ_s = soil unit weight

B = cone diameter

N_{rq}= Bearing capacity factor for wedge penetration (plain strain)

ξ_{rq}= shape factor to convert wedte factors to cone factors.

Therefore, the value of N_{rq} , ξ_{rq} can be calculated from the above equation, which is dependent on soil friction angle ϕ' , base roughness δ / ϕ', relative depth of penetration D/B, lateral earth pressure coefficient K_o' and cone apex angle 2α Durguoglu and Mitchell (1975) have presented charts to calculate N_c, ξ_c, N_{rq} , and ξ_{rq} and thereby ϕ' and c can be estimated indirectly

Schmertmann Method (1975)

This is an indirect method for estimating ϕ' through the relative density (D*r*) parameter From the results of chamber tests on normal consolidation, medium to fine dry and nearly saturated sands carried out at the University of Florida, Schmertmann constructed the curves shown in Figure 2 8 (a) which are also based in part on the results from the relative density studies of Mississippi River sands below water table. After making estimate of D*r* from Figure2.8 (a), the value of ϕ' can be estimated using the correlation given in Figure 2.8 (b)

Overconsolidated sands must be converted to their equivalent normally consolidated q_c before entering Figure2.8 Following two equations can be used for this purpose if an independent estimate of the overconsolidation ratio (OCR) or the insitu K_o' coefficient of the sand is known

$$\frac{K_O'}{K_{ONC}'} = (OCR)^{0.42}$$

$$\frac{q_c}{q_{cNC}} = \left(1 + 3/4\left\{\frac{K_O'}{K_{ONC}'} - 1\right\}\right)$$

where OCR = overconsolidation ratio

K'_o = lateral pressure coefficient

Mitchell and Lunne (1978) applied this method to various CPT results and found that the value of ϕ' estimated compare well with other methods

2.4.1.2 Advantages and Disadvantages of these methods

Each method has its own merits and demerits and is applicable to a certain type of soil. De Beer's theory has been used widely, although it is now recognised that it gives very conservative value of ϕ'. Meyerhof's method gives ϕ' values reasonably well where q_c does not increase significantly beneath a certain depth i e. in case where critical depth is found. Further disadvatage of Meyerhof theory (1961) is that it can be used only for granular soils having $\phi' > 34°$. Jambu and Senneset method compares reasonably with actual field measurements However, the application of this theory requires a straight line plot of cone pressure vs depth profile Quite often it is difficult ot interpret straight line profile from actual field results. Tofimenkov method is simple to apply provided the overburden pressure does not exceeds 1 Kg/cm². This is a serious limitation of this method and restricts its use to shallow depths only Durgunoglu and Mitchel method is based on laboratory penetration test results and gives considerable high value of ϕ. Further the interpretation in this method requires a lot of mathematical calculation to arrive c and ϕ values Schmertmann method is easy to apply for interpretation of the penetration test results of normally consolidated, medium to fine dry sands. In case of overconsolidated sands, this method requires additional information such as overconsolidation ratio and in-situ lateral pressure coefficient (K_o) for converting q_c into $q_{c\,NC}$

2.4.2 Standard Penetration Test

The standard penetration test, developed in 1927, is currently the most popular and economical means to obtain subsurface ingormation The method has been standardized as ASTM D 1586 since 1958 with periodic revisions to date The same has been standardized by Indian Standards too Prior to and after ASTM standardization it was regularly observed that N values in adjacent boreholes were not reproducible. Because of wide SPT use this problem received much attention -- first by Gibbs and Holtz (1957) who considered overburden pressure and length of drill rod were the principal causes of nonreprducibility Beyond this not much was done until de Mello (1971) presented a comprehensive leterature survey which started a focus on the driving energy {Schmertmann (1975)}

Conventional practice is to do an SPT every 1 or 2m after penetrating the topsoil or starting the first test at about 1 to 1½ m depth. For each test there is a sample recovery of about 460 mm including the seating depth to plrovide a visual profile of around 50 percent of the boring depth

Terzaghi and Peck (1967) and Peck et al (1974) have proposed empirical correlations between the penetration resistance N, allowable bearing pressure and other soil properties such as density index and angle of shearing resistance,etc. A correlation of penetration with the angle of shearing resistance ϕ given in Figure 2.9. Studies indicate that the penetration resistance reflects both the insitu density index and the effective stress to the depth of testing ; therefore a number of combinations of stress level and density index may result in the same measured N value To arrive at a standardized value of N for use in empirical correlation, an overburden pressure correction is necessary For a constant density index, the N value increases with the increasing effective overburden pressure for which corrections have been proposed by Gibbs and Holtz (1959), Thornburn (1963) and others.

2.4.3 Plate Load Test

Plate loading tests may be performed either to determine the ultimate bearing capacity of a material or to establish relationships between load and deformation. Terzaghi and Peck (1948) give an excellent treatment of the conditions under which load tests are suitable, the requirements of the testing procedures, and the analysis of the test results The usual practice is to load small plates of diameter 12 inches or square of side 12x 12 inches Ultimate bearing capacity is found from the load settlement curve

The bearing capacity and load deformation modulus of granular soils are influenced by surcharge pressures due to the weight of material above and adjacent to the bearing area. If a plate bearing test is to be made below the existing ground surface and unless the prototype footings are to be favoured by a significant depth of surcharge, it is necessary that the test pit be sufficiently large to minimize the effects of the surcharge in the bearing test. Tezaghi and Peck (1948) recommend that the pit be at least 5 times larger than the size of plate in sand

To determine the strength parameters, plates of several sizes (at least three) are used from which ultimate bearing capacity of each of the plate is calculated The bearing capacity factors N_c , N_q and $N\gamma$ are calculated from these bearing capacity equations Since our discussion is restricted here for cohesionless soil, therefore only N_q and $N\gamma$ are required to be calculated for the evaluation of angle of internal friction 'ϕ'. The correlation between the angle of internal friction 'ϕ' and bearing capacity factors is shown in Figure2.9 as given by Peck, Hansen and Thornburn 1953 which automatically incorporates local shear failure.

Table 2.1 : Chemical Composition Indian Ashes and Ash of Other Countries

Chemical Constituents	Indian Ash	Indian Fly Ash	Swedish ash	U K Ash	U S Ash
SiO_2	47 0-65 3	48 41-70 5	30 0 -53 0	38.0-58 0	30.0-58 0
$Al_2 O_3$	16.0-32 0	16 1-34 1	14 0-33 0	20 0-40 0	7 0-38 0
$Fe_2 O_3$	3 8-17.1	1.7-19 6	10 0-14.0	6 0-16.0	10 0-42 0
CaO	1 1-5 3	0.56-11.87	0.9-6 1	2 0-10 0	0 0-13 0
MgO	0 5-2.4	0.1-3 68	4 0-6.0	1.0-3.5	0 0-3.0
SO_3	0 1-3 2	0 1-2 95	1 6-3.5	2 0-5 5	0 4-2.0

Table 2.2 : Range of Specific Gravity

Material	Specific Gravity
Bottom Ash[1]	2 28 - 2 78
Fly Ash	
USA[2]	2 03 - 2 67
India[4,5,6]	1.84 - 2.67
Pond Ash	
India[4]	1 58 - 2 39
UK[3]	2.10 - 2 24
Poland[3]	1 90 - 2.31
Soils	2.60 - 2.80

1	Seals et al (1972)
2	Martin et al (1990)
3	Skarzynska et al (1989)
4	Dayal et al (1989)
5	Rajasekhar et al , 1995
6	Sivapullaiah et al , 1995

Table 2.3 : Range of values for Particle Sizes (Sridharan,1995)

Material	Effective Size d_{10} (mm)	d_{60} (mm)	Uniformity Coefficient	Coefficient of Curvature
Fly ash	0.0027-0 02	0.036-0.2	13 3-10.0	2 16-0 73
Pond ash	0.014-0.038	0.1 - 0.2	7 1 - 5 3	1.20 - 0.95
Bottom ash	0 08 - 0.7	0.46 - 6.4	5.8 - 9 1	0.83 - 1.03

Table 2.4 : Maximum Dry Density And Optimum Moisture Content

Class of ash	Mean Dry Density (gm/cc)	Optimum Moisture content (%)
Fly ash	1.21	29 8
Coarse ash	1 34	25 8
Combined ash	1.3	26.8

Table 2.5 : Values of Coefficient of Permeability for Ashes

Material	Coefficient of permeability (cm/sec)	Reference
Lagoon Ash		
Poland	1.5×10^{-4} to 5×10^{-5}	Skarzynska et al., 1989
India	4.6×10^{-4} to 1×10^{-4}	Dayal et al., 1988,1989
Indian fly ash	10^{-4} to 10^{-6}	Rajashekhar, 1995
British fly ash	8×10^{-5} to 5×10^{-7}	Gray and Yen-Kuang Lin, 1972
Swedish fly ash	10^{-5} to 10^{-7}	- do -
Western fly ash	1.65×10^{-5}(ave.)	Dayal et al., 1988,1989
ASTM class C fly ash	2×10^{-7} to 1×10^{-7}	Yudhbir & Honjo, 1991
ASTM class F fly ash	6×10^{-5} to 2×10^{-6}	- do -
Bottom ash	3.6×10^{-4} to 2.4×10^{-4}	Dayal et al., 1988, 1989
Western bottom ash	3×10^{-2} to 3×10^{-4}	- do -
Gravel	10 to 1	- do -
Sand	1 to 10^{-3}	- do -
Silt	10^{-3} to 10^{-7}	- do -
Clay	$\times10^{-7}$	- do -

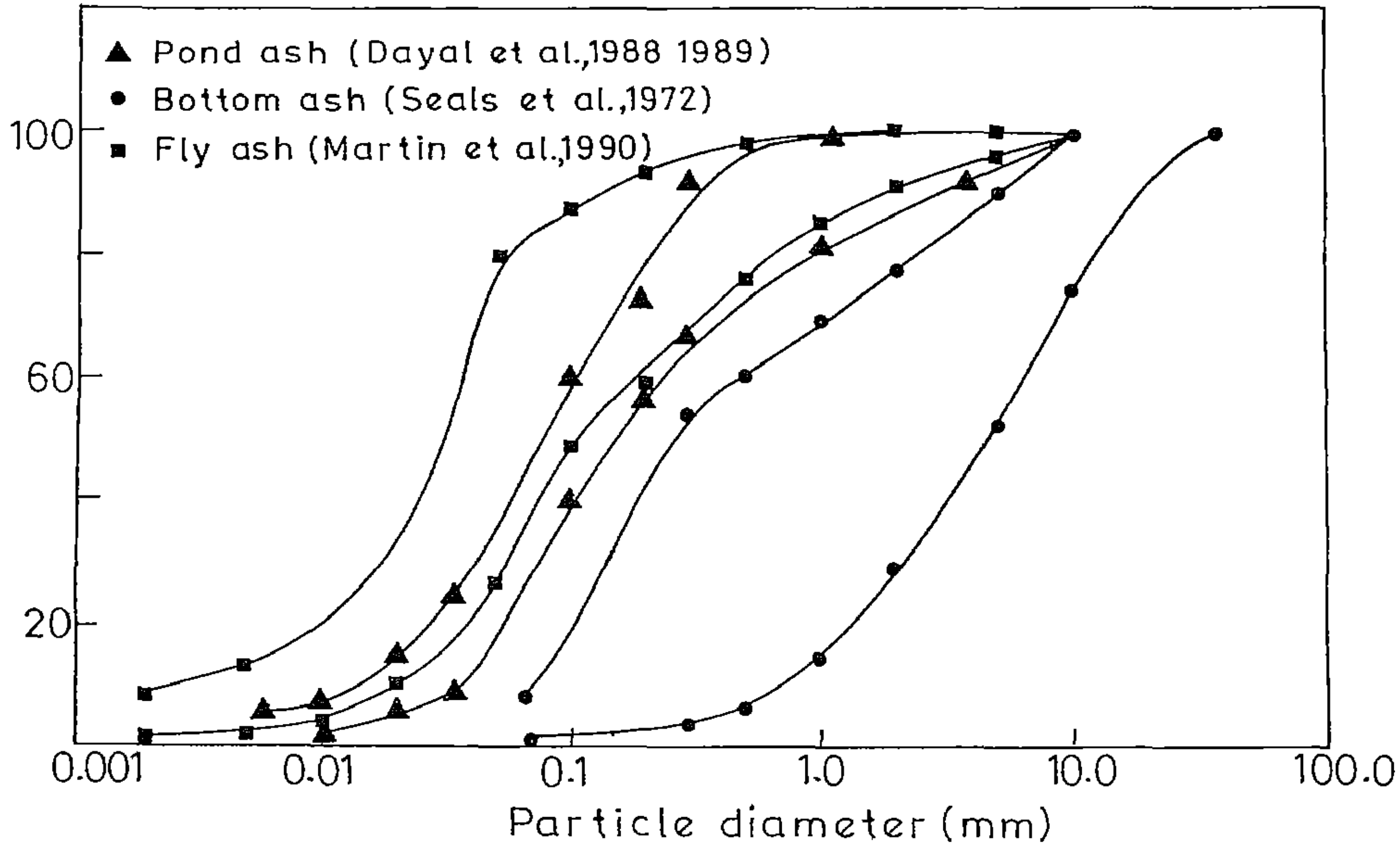


FIG.2.1 TYPICAL GRAIN SIZE DISTRIBUTION CURVES FOR COAL ASH

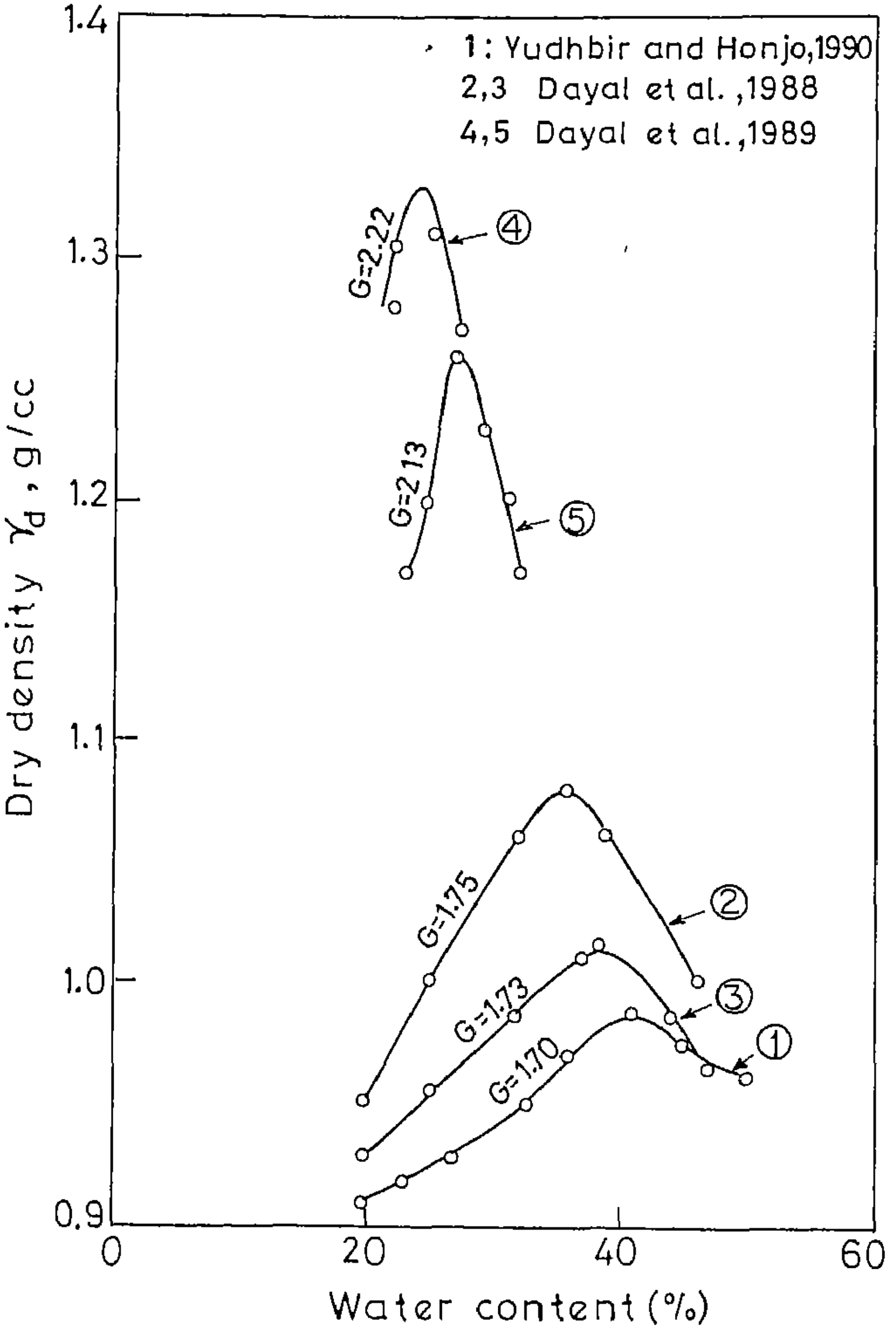


FIG.2.2 COMPACTION CHARACTERISTICS OF POND ASH

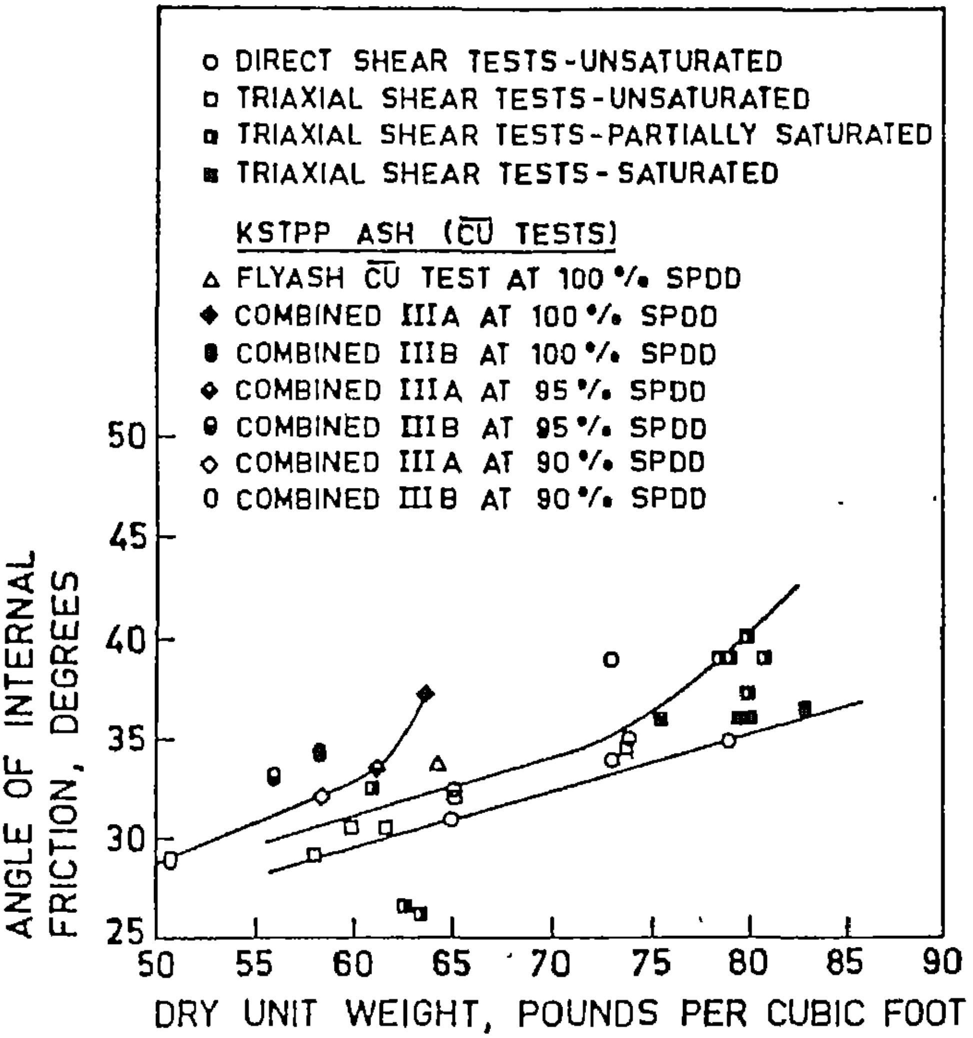
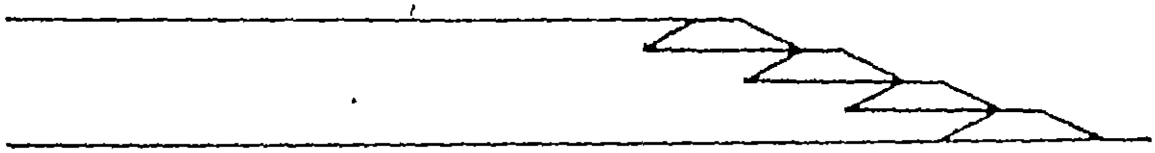
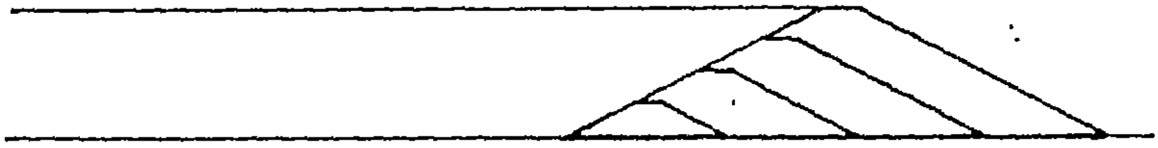


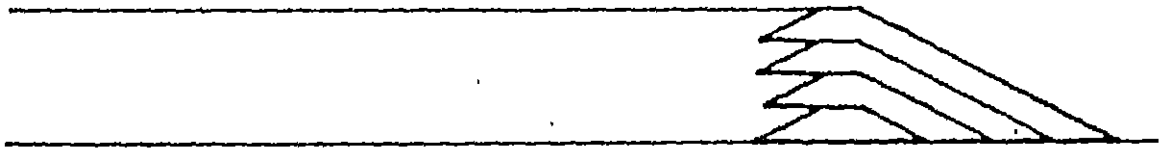
FIG. 2.3 ANGLE OF INTERNAL FRICTION – DRY UNIT WEIGHT RELATIONSHIP FOR BITUMINOUS FLY ASH (AFTER DIGIOIA AND GRAY (1979).



(a) Upstream method

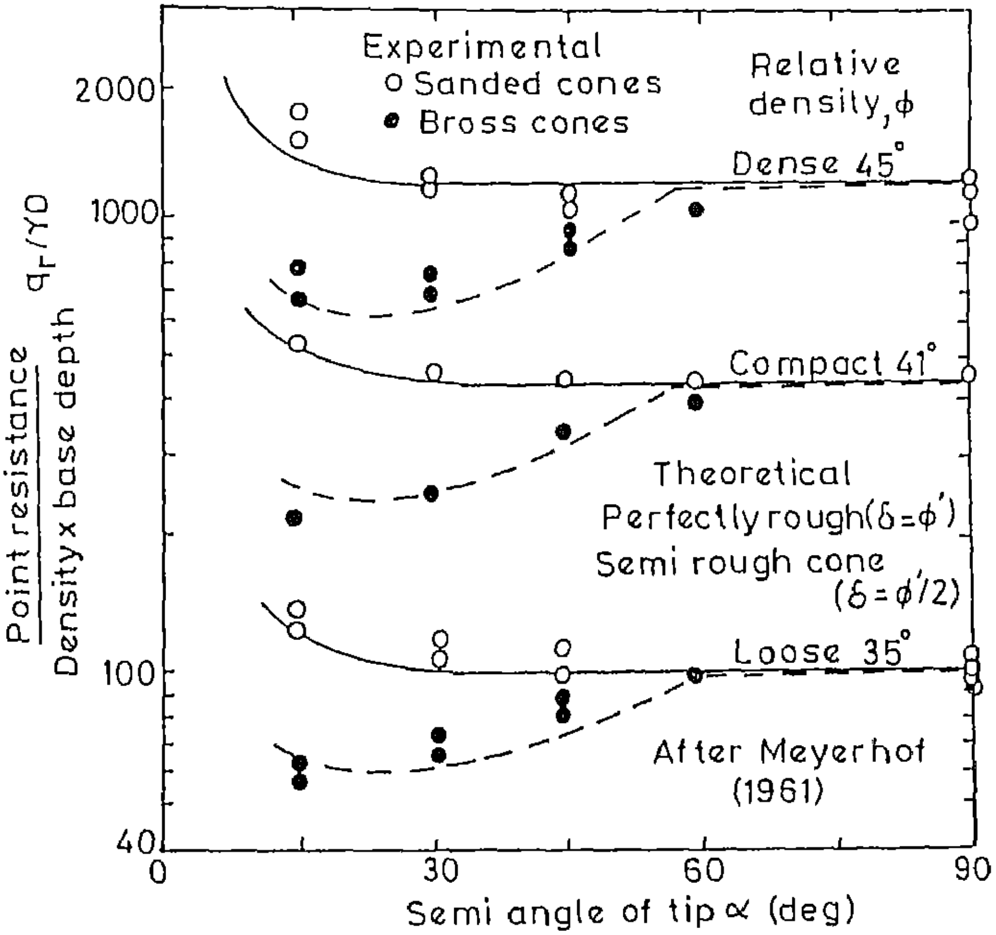


(b) Downstream method

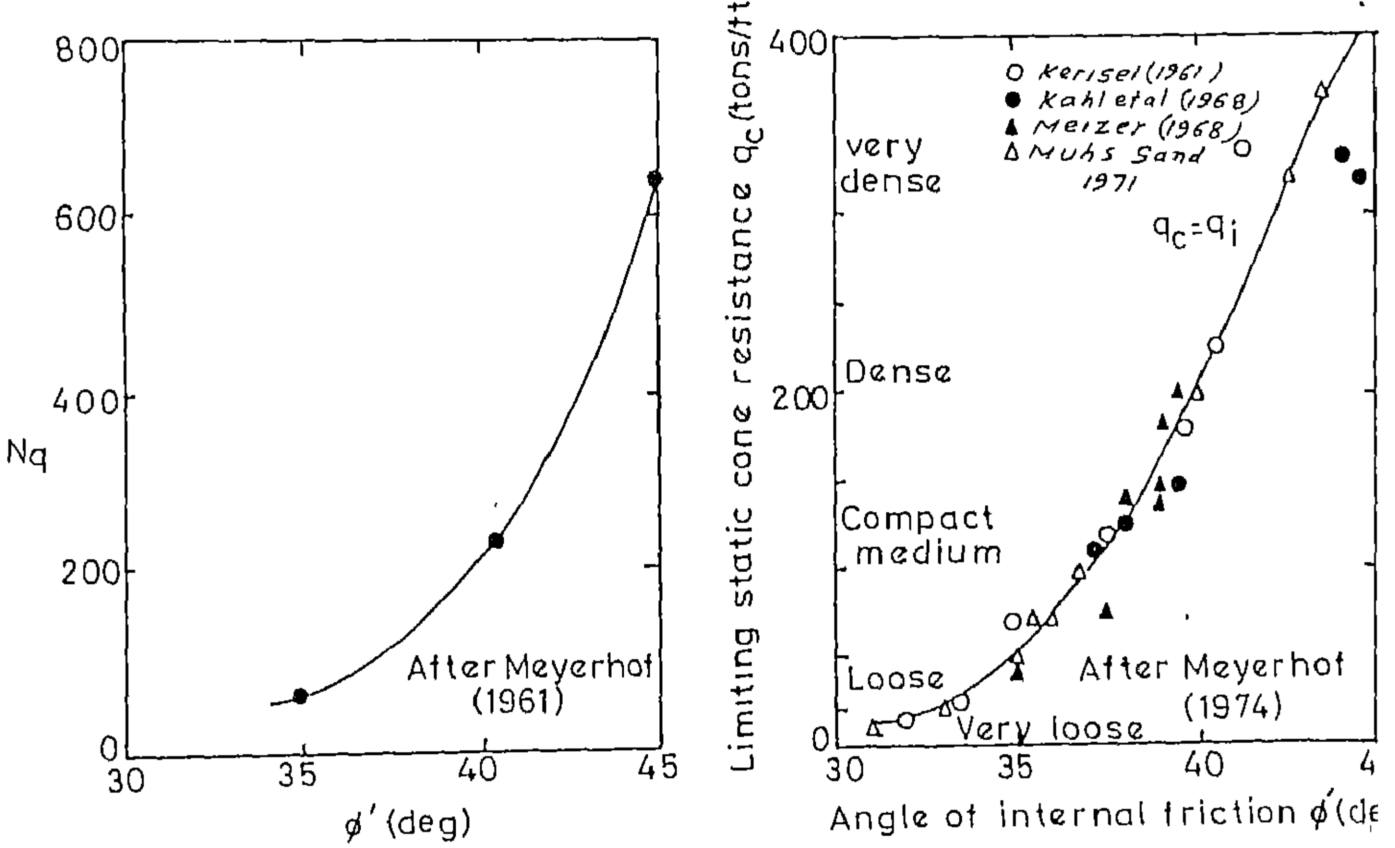


(c) Centreline method

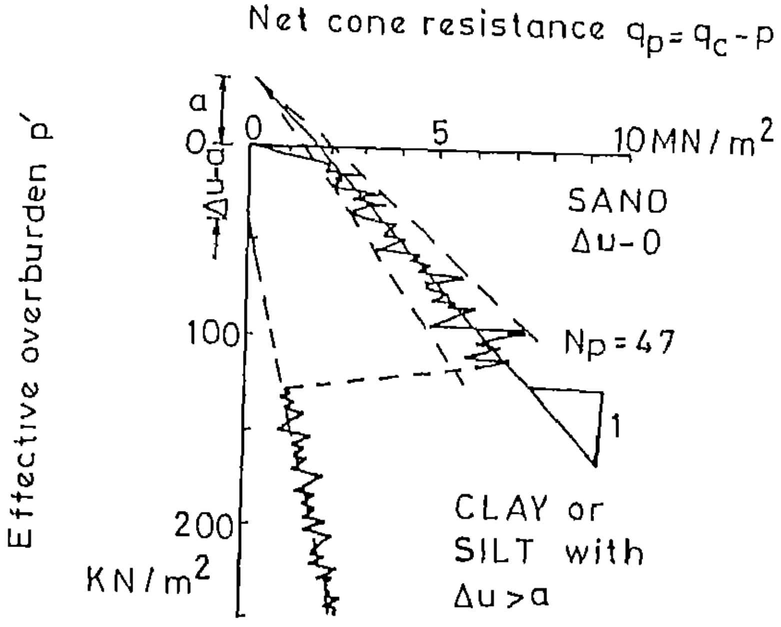
FIG.2.4 METHODS OF RAISING DYKE HEIGHT



(a) Effect of roughness and apex angle



(b) Nq for 30° semi apex (c) Limiting cone resistance



(a) Principle of interpretation

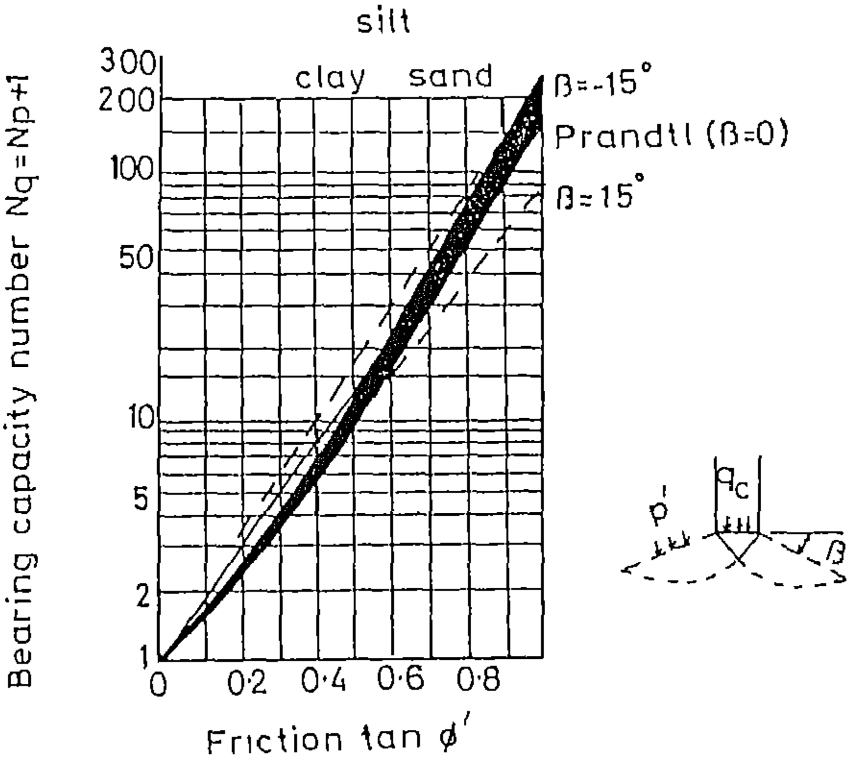


FIG 2.6 JANBU & SENNESET METHOD FOR DETERMINING φ′ AND a FROM CPT SOUNDING LOG

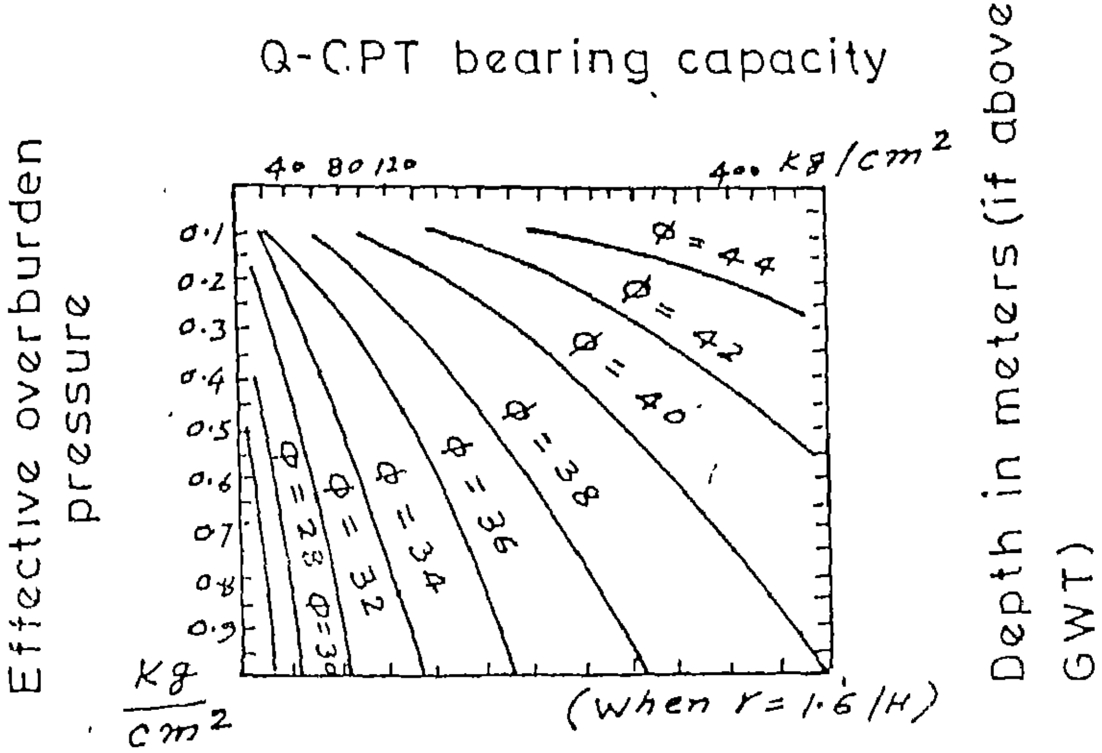
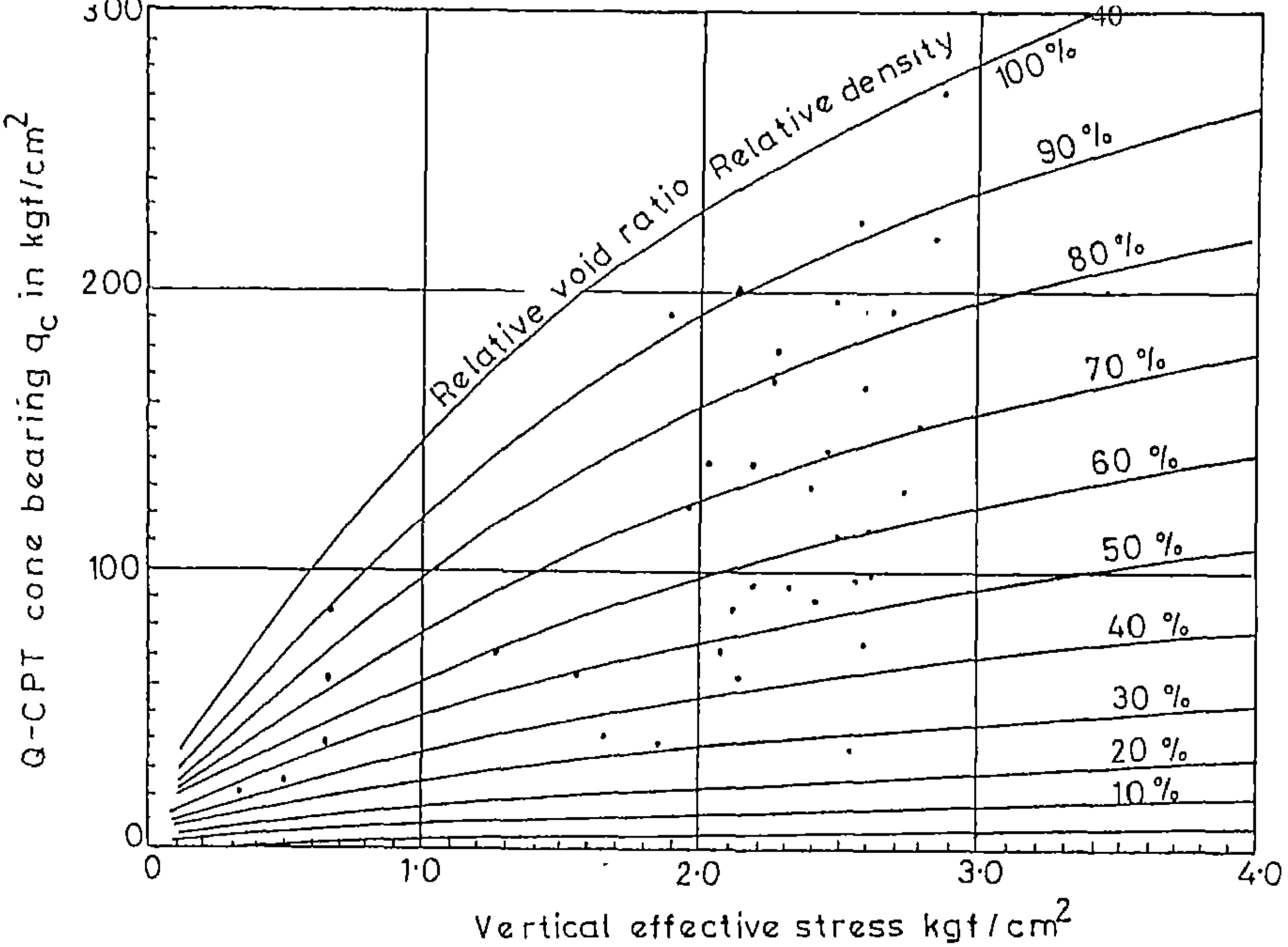


FIG. 2.7 METHOD FOR ESTIMATING ϕ' FROM q_c REPORTED IN USE IN USSR (Trotimenkov method, 1974)



IG 2.8 a Q-CPT BEARING CAPACITY TO ESTIMATE RELATIVE DENSITY IN NORMALLY CONSOLIDATED SILTY FINE TO UNIFORM MEDIUM SANDS, (Schmertmann 1975)

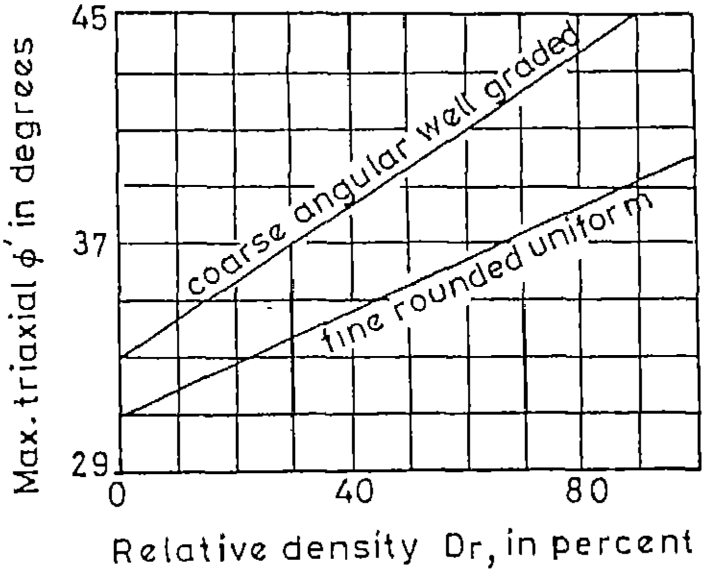


FIG.2 8 b. APPROXIMATE CORRELATION BETWEEN ϕ', AND D_r IN QUARTZ SANDS, (Schmertmann 1975)

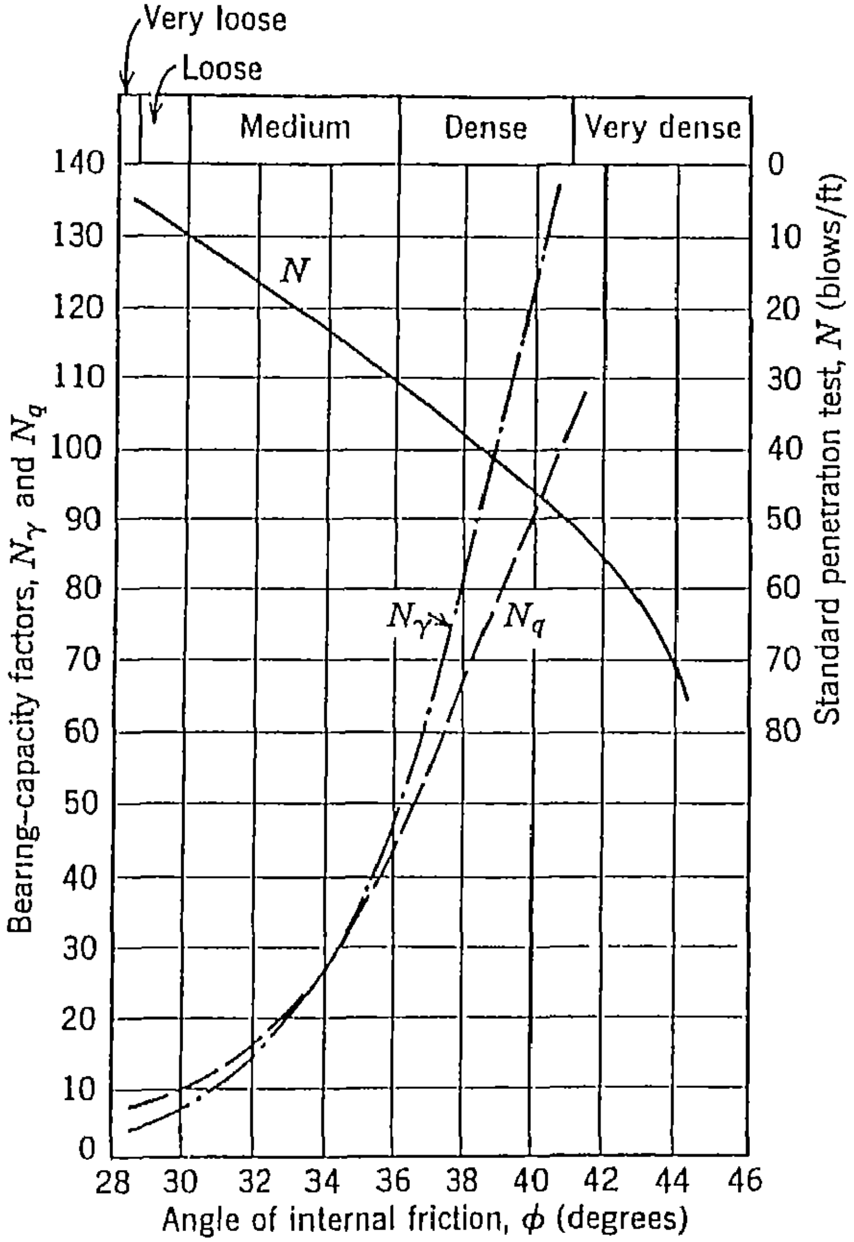


Fig. 2.9 Bearing capacity factors which automatically incorporate allowance for local shear failure (From Peck, Hansen, and Thornburn, 1953).

2.5 Pollution Factors

Land is the primary and most essential resource for sustenance of living organisms in the biosphere The cultural evolution of a man an important component of biosphere has led to the utilization of natural resources by industrialization and urbanization In these ventures, injudicious planning has ultimately resulted in serious environmental hazards jeopardizing man's own existence Development programmes and unabated urban growth as a consequence of fast development of industrial activities place an increasing strain on the infrastructural facilities as well as air, water and land resources in the country With increasing industrialization, pollution control by various means is going to be a major concern and challenge for technologists of twenty first century. The production of energy by thermal power plants (TPPs) ranks at the top of the pollution intensive industries The pollutants generated in this industry contaminate air, water, and biota in the zone of influence which ultimately find its respite on the land.

Two major sources of pollution can occur from ash disposal area installations which is air borne and water-borne

2.5.1 Air-Borne Pollution

Air borne pollution is caused by wind and traffic movement both from the mound and from any unsurfaced access roads The construction area in case of ash mound is particularly vulnerable until protection on landscaping is completed Air borne dust resulting from wind blowing over unprotected waste embankments and dried slurry material can be harmful to vegetation The transportation and deposition of fly ash can create a considerable amount of fine particulate matter that is susceptible to wind erosion. Exposed fly ash is also subjected to compaction and crushing by machinery and is further deteriorated by physical weathering and chemical decomposition Fugitive ash was the major problem in Korba

Mitigatory Measures :

There are certain precautionary measures which has be considered before deciding the location of the fly ash dyke These are :-

- The location of Ash pond/mound is such that the predominant wind direction is towards the ash disposal area, unlike Korba where the whole of the fugitive ash blows towards the city from NTPC's ash dyke.
- Ash disposal area to be located away from inhabitation as far as possible

There are certain methods which are although a time consuming process but still are the most frequently used methods, for the prevention of ash to be air borne such as listed below :-

- Establishing wind breaks of trees or hedgerows that alter both the direction and velocity of wind over the ash pond
- Providing ground cover by planting wild grass to arrest the ash being air-borne

In situations requiring a relatively quick dust control procedure or where vegetation is for some reason impractical, stabilization has been achieved using various commercial chemical agents. Chemical seals have been obtained through :

- The spreading of a lime chip sodium silicate or potassium silicate topdressing over the ash pond.
- The use of a resinous or bituminous- base adhesive
- The use of calcium, ammonium and sodium lignin-sulfonates and bark extracts
- Resin and wax emulsions or neoprene
- Elastomeric organic polymers (Coalgate, et al ,1973 ; Dean and Havens, 1972 ; and Eigenbrod, 1971)

But there are then certain methods which are faster as well as easy preventive measures than listed above such as :-

- Providing water-cover over the filled pond area, as done at KSTPP ash dyke and BTPS ash dyke in case of active pond.
- Water sprinkling system during the construction of ash mound This method was recently being used in Korba, where the slurry discharge outlet was covered by a plate with holes arranged in such a way that it formed several jets of ash slurry and thus in this way the nearby portion of pond was always kept moist
- Applying a stabilizing layer of soil or bark mulch over the pond.
- Disking straw into the top few inches of the ash pond.

2.5.2 Water Borne-Pollution

Water borne pollution is caused either due to soluble contents from fly ash or through leaching action to subsurface water and also suspended solids through its effluents Although a variety of water quality problems can be created by fly ash drainage, the most common types of pollution are :(i) increased turbidity and suspended solids, and (ii) water quality degradation due to acidic leachates The water pollution problems created by some fly ash can be very serious. Fly ash leachates are most often quite acidic, usually contain elevated concentrations of trace metals such as iron , aluminum and manganese and are also quite corrosive Since it also contains As, Pb and Hg which are harmful for human population.

2.5.2.1 Turbidity and suspended solids

Inadequate filtration techniques can create heavy loads of suspended solids in seepages and drainages from ash dykes which degrades aquatic ecosystems When these solutions reach streams and/or lakes, turbidity is increased and light penetration is reduced, thus lowering aquatic food production Much of the suspended material eventually settles out of solution in the form of silt, and thereby decreases both the size and the quality of the aquatic habitat

However, since total prevention of erosion is difficult, additional procedures have been developed to control the amount of suspended solids reaching streams

Prevention -

Any time soil and other earthen material are exposed to weathering, erosion and siltation occur The following measures can be implemented to minimize their occurrence

- Location of ash pond/mound area having low ground permeability, as far as possible
- Provision of graded-filters in decanting towers for filtration of ash pond effluents.
- Turfing of Ash-dyke to prevent seepage from ash pond.
- The stripping of vegetation from a disposal site should be limited to only that area which is immediately needed for construction. Future fill areas should be stripped only as they are needed ;
- Topsoil that is removed for the construction are stockpiled for future use in planting should be stored in a manner that minimizes erosion ;
- During the construction process, care should be taken to preserve vegetation on areas peripheral to the disturbed construction area ,
- Temporary collection ditches leading to temporary settling basins should be constructed at the downstream end of the construction site ; and
- All exposed fill material should be graded in a manner that discourages runoff over the downstream face of the embankment

Control-

As indicated above, the prevention of all erosion and siltation during construction is difficult Control procedures have therefore been developed for reducing the amount of suspended material entering streams These procedures include ·

Settling Basin - Settling basins are structures designed to intercept and retain water borne sediment and debris They are primarily intended for use during a construction operation and prior to the establishment of vegetation on the disturbed area These catchment structures are normally not intended to retain water for long periods and are usually dry except after rainstorms

Erosion Control Netting - Erosion control netting can be used to stabilize freshly seeded drainage or diversion ditches, until such time that a cover of vegetation is established. Once these ditches are seeded and mulched, a biodegradable netting material can be placed along the ditch surfaces and stapled in place This netting holds the mulch, seed and soil until after germination and the establishment of a root system.

Vegetation - Erosion and stream turbidity are best minimized by establishing a protective layer of vegetation on embankment slopes and along exposed ditch surfaces. The establishment of grasses in drainage ditches reduces their flow velocity and thereby protects the interior faces form erosion

Chemical precipitation - In some instances where very fine particles of material are suspended , the amount of time required for natural settlement or clarification in a settling basin is quite lengthy In such cases, clarification can be accelerated using chemical flocculents such as high density polyelectrolytes. By altering the electrical balance of the suspended material, particles are attracted to each other and form aggregated that are too large to remain in solution Both types of precipitate rapidly settle to the bottom of the settling basin and form a sludge like deposit. Continuous sludge removal is required during this type of operation in order to sustain the capacity of the system

2.5.2.1 Acidity

Acid drainage is probably most threatening water quality problem associated with coal ash Waste constituents such as Fe, Al, Ca, Cl, SO_4 and SO_3 are not generally regarded as hazardous They are included in secondary drinking water standards But the concentration of As, Cd, Cr, Hg, Ni and Pb as low as 0.01 mg/l can have serious effects on certain aquatic species. The standards laid by Central Pollution Control Board for ash pond effluent are -

pH	6 5 - 8 5
suspended solids	100 mg/l
oil and grease	20 mg/l

The presence of trace metals like arsenic, cadmium, vanadium etc. even though in very small amounts can be very harmful to human population as well as to plants and animals. Being highly toxic, these metals can cause severe poisoning and several other health hazards to human habitations located around the thermal plants burning coal. The water though ash pond can cause leaching of heavy metals into the subsurface and ground water in and around ash pond. When leachates percolate into the ground water, aquifers can become polluted and thus drinking water obtained from these sources can create health problems

In order to protect nearby streams, the acid leachates must be treated after they leave an embankment, but prior to entering the natural drainageways Although many methods for treating acid mine drainage exists, only a few have been extensively used for coal ash. These have all involve, in one form or another, the neutralization of the acid drainage. These experimental treatment methods include · reverse osmosis, dilution , sodium treatment , microbial inhibition, hgh energy radiation, distillation, ion exchange, foam separation, use of latex, freezing and some ozone ixidation

The more common method of treating acid drainage is by neutralizing the leachates. There are many alkaline reagents that can be used to neutralize acid drainage such as listed below .

Table 2.6 : Relative Efficiency of Reagents

Reagent	Chemical Formula	Relative Efficiency
Limestone	$CaCO_3$	0.80
Quick Lime	CaO	0.90
Hydrated Lime	$Ca(OH)_2$	0.95
Caustic Soda	$NaOH$	0.99
Soda Ash	$Na_2\,CO_3$	0.99

The above efficiency ratings are dependent upon a number of factors, however, the principal determinant is the solubility of the reagent in water. If enough neutralizer is added to the drainage, the solution may even become alkaline and free of most harmful metals

2.5.3 Suggested Measures For Pollution Control For Ash Ponds And Ash Mounds

Apart from the measures taken to minimize the pollution level in ash disposal areas, there are many schemes/proposals being sought for new power projects. The important provisions are .

a) *Recirculation of Ash Pond Effluent* ·

The ash pond effluent shall be recirculated for the use inplant and disposal of ash in wet conditions. If recirculations is adopted the overflow of lagoons will be treated suitably and hence, there may not be any need for providing filter ponds. This will conserve water for plant use.

b) Ash Pond Lining

There is no definite consclusion available that the leachates of Indian ash is polluting the ground water. THE chemical analysis of ash from various NTPC projects, indicate that the amount of harmful elements present in the effluent of ash pond are far below the allowable limit.

The provision of lining may be based on geological conditon of area as well as result of monitoring of effluents

c) Sprinkling of WTP Sludge on Ash Pond ·

This is an innovative measure taken by Badarpul Thermal Power Station of NTPC by sprinkling the water treatment plant sludge on completed surface of ash pond This has resulted into 3 fold advantage

I Controlled the fugitive dust emission by forming a thin film on the finished surface of ash

II Enhanced the fertility of ash for grownth of vegetation and ground cover

III The disposal of WTP sludge, otherwise a waste material, is dispensed-with

d) Afforestation

Plantation of saplings in abondoned/completed ash pond area is an important measure of pollution control For the improvement and enriching the ash surface in order to propogate and maintain vigorous vegetation growth, the selection of species is an important criteria

e) Landscaping

To ensure long term stability and to give an environmentally acceptable appearance to the ash mound the surface is to be landscaped with appropriate vegetation after trial.

2.5.3.1 Monitoring Systems

To control the parameters causing pollution ist is imperative to have an effective monitoring systems. Following parameters are monitored regularly for ash management systems

- Fugitive Dust Emission Level
- Ambient Air Quality
- Ambient Water Quality
- Ash Pond Effluent

Chapter 3

Instrumentation

3.1 General

There are several varieties of geotechnical instruments used for monitoring of the performance of structures In case of earth dams the primary concern is the horizontal, vertical movements and pore water pressure built up in the body of dam. In order to monitor the performance of ash dyke during construction and operation, the following instruments were installed at Korba Super Thermal Power Plant ash dyke

1. Settlement Plate
2. Piezometer
3. Inclinometer

3.2 Settlement Plate

Settlement Plate has been installed to measure the consolidation of the lagoon ash below the crest of the ash embankment. The device is designed to measure the consolidation settlement of the settled ash and the total settlement of the embankment.

The settlement plate unit consists of a base plate section, an anchor cross arm section, and pipes of diameter of 0.04 and 0.05 m pipe. A typical installation detail is shown in Figure 3.1. The base plate section consists of 0.075 m medium weight steel channel to which two flat plates are welded. This channel is then bolted and welded to a length of 40 mm diameter standard steel pipe. This unit is then nested in a section of 50 mm standard steel pipe anchored into the lagoon ash. An anchor cross arm is attached to the 50 mm pipe, 0.10 m above the base plate. The base plate is installed on a prepared surface at an elevation at which the settlement is desired. At the center of the plates a steel riser pipe or rod is connected. As the fill is constructed, the riser is extended, with the top kept a few feet above or below the surface of the fill, as may be required by job conditions, to prevent damage to the riser by equipment to measure the consolidation of the embankment. This entire apparatus is installed as the embankment is constructed. Readings are taken by lowering the reading probe into the 50 mm pipe and reading the distance to the base plate. The elevation of the top of pipe is also determined. The readings are taken at intervals of one month for firsts six months after the construction and thereafter at two months intervals using theodolite.

The construction sequence operations are

- After removal of ash from the lagoon to 0.5 m below the settled ash, a hole of at least 75 mm is drilled to a depth of 1.5 m
- Clean pea gravel or river gravel of 6 to 10 mm size are placed to a depth of 0.15 m in the bore hole.
- PVC pipe is placed in the bore hole with perforations at the bottom.

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- Area around the pipe is backfilled with 6 to 10 mm clean pea gravel or river gravel until the material covers the perforated length of 1 0 m of the pipe
- 0.25 m Bentonite plug is placed by pouring dry bentonite balls around the pipe and tamping with a steel rod

The magnitude of the foundation settlement of an embankment depends on the height (weight) of the embankment, the depth and thickness of the compressible stratum within the foundation, and the compression indexes of the foundation material. From these observations a time settlement record of the plates is obtained

3.3 Piezometer

The long term stability of a slope, to a large extent depends on the pore water pressure which develops along potential sliding surface Piezometers are used to measure the hydrostatic or excess water pressure in the pores of saturated soils Information on these neutral stresses is important in the study of bearing capacity, earth pressures, settlement and stability of slopes There are four basic types of piezometers available commercially , open stand pipe, hydraulic, pneumatic and electronic Since fly ash generally have high permeability therefore open pipe stand type of piezometer is quite suitable and most economical It works on the principle of observation wells. The piezometer consists of plastic pipe which extends from the tip elevation shown in the Figure 3 2 to the top of fill

The water level in the stand pipe can be measured to an accuracy of 0.01 m using several devices, including the plumb bob, cloth or metal surveyor's tapes coated with chalk or commercially available electrical indicators. In order to monitor the piezometric level within the fill in KSTPP ash pond full perforated pipe piezometers of 38 mm diameter have been installed and electrical indicators are used to measure the water level in piezometer An electric indicator consists of a probe which is inserted through the open stand pipe and as soon as the probe meets the water level the sound box gives an alarm The piezometer level is generally taken at one week interval

3.4 Inclinometer

Inclinometers are used to monitor lateral earth movements in land slide areas and embankments They can also monitor deflection of retaining walls and piles. Horizontal inclinometers are used to monitor settlement in foundations and embankments. An inclinometer system includes inclinometer casing, an inclinometer probe, control cable and inclinometer readout unit. The inclinometer used during the investigation was manufactured by Slope Indicator company, a Boart Longyear group company, Seattle, U S A

3.4.1 Inclinometer Assembly

3.4.1.1 Inclinometer Probe

The Digitilt inclinometer probe is precision device The inclinometer probe is used to survey the casing and establish its initial position. Probe does not measure the displacement directly. Instead, it measure the inclination (tilt) of the casing Displacement is calculated from changes in inclination. Inclination is measured by two force-balance accelerometers One accelerometer measures tilt in the plane of the inclinometer wheels (the " A "axis) The other accelerometer measures tilt in a vertical plane perpendicular to the wheels (the " B" axis) During the survey, the probe is drawn upwards from the bottom of the casing to the top. It is halted at half-meter intervals for inclination measurements.

3.4.1.2 Casing

The casing used with inclinometer is made up of Aluminum with Epoxy coating over it. The casing has a outside diameter of 3.5 inches (8.89 cm) and inside diameter of 2.875 inches (7.3 cm) It consists of four grooves perpendicular to each other. These grooves helps the wheel of the inclinometer probe to travel in the required direction These grooves are termed as A0, A180, B0 and B180 and their directions are shown in Figure 3.3 (c). A0 is used as a reference groove and should be marked on the casing. Inclinometer casing is typically installed in a near vertical bore-hole that passes through a zone of suspected movement The bottom of casing is anchored in stable ground and serves as a reference. Ground movement causes the casing to move from it's initial position to a new position The rate, depth, and magnitude of this displacement is calculated by comparing data from the initial survey to the data from subsequent surveys

3.4.1.3 Control Cable

Digitilt control cable is used to control the depth of the inclinometer probe. It also conducts power and signals between the probe and the readout It is durable and easy to handle and the cable stays flexible in extreme temperatures and resists chemicals and abrasions. A steel strain wire and special binders provide excellent dimensional stability. Flexible rubber depth marks are permanently vulcanized to the cable jacket. The marks cannot loosen and slide along the cable and have no rigid edges that can damage the cable jacket and conductors Metric cables are graduated with yellow marks at 0 5 m intervals and red marks at 1 m interval.

3.4.1.4 DataMate

The Digitilt DataMate is a compact inclinometer readout that record data from a Digitilt inclinometer probe The DataMate stores over 10,000 data points in up to 40 data sets. The DataMate Manager software (DMM) is an integral part of the DataMate system DMM creates a project database that holds records of inclinometer installations and data from inclinometer surveys and is used to transfer data from the DataMate to the project database on the computer. DMM can also generate simple reports and graphs

3.4.1.5 Digipro

Digipro designed to speed the handling and interpretation of inclinometer data It can perform sophisticated data manipulation and produce outstanding graphs such as displacement (movement), deviation (profile) and displacement vs time It can also generate and rotate three-dimensional graphs.

3.4.2 Data Recording and Display

During the survey, the probe is drawn upwards from the bottom of the casing to the top It is halted at half-meter intervals form inclination measurements A full data set includes both 0 and 180 degree readings for each axis The sign of the 180 readings is opposite that of the 0 readings since the probe is rotated 180 degrees Ideally the two readings for each depth would be identical except for the sign, but sensor offset and sensor placement introduce differences To eliminate offset errors, the probe is drawn through the casing twice On the second pass, the probe is rotated 180 degrees During data reduction, the 0 and 180 degree data are merged Therefore, the two readings are averaged by finding the algebraic difference of the two values and dividing by two. The dataset thus recorded is transferred to the computer which reproduce the following types of graphs

- Cumulative Deviation
- Incremental Deviation
- Cumulative Displacement
- Incremental Displacement

Deviation is calculated from the tilt of the casing . Incremental deviation is the deviation that occurs at a single interval. Cumulative deviation is the sum of incremental deviations The graphs of cumulative deviation show the profile of the casing referenced to a fixed point at the bottom or top of the casing The plotted point at any interval is the sum of incremental deviations up to and including that interval The fixed point appears at the bottom of the plot with a valve of zero

Displacement is the distance that the casing has moved A graph of *displacement* shows *changes* in the position of the casing. In contrast, graph of deviation shows the actual position of the casing. Displacement is always calculated by comparing a "current" data set to an "initial" data set. Digipro subtracts the initial deviation at each interval from the current deviation at the same intervals. The result is called *incremental displacement*. The sum of the incremental displacements is called cumulative displacement

$$\text{Cumulative Displacement}_n = D_1 + D_2 + D_3 + \ldots + D_n$$

where D= Deviation

The incremental displacement graph makes it easy to find the zone where maximum movement is occurring

$$\text{Incremental Displacement} = D_{current} - D_{initial}$$

3.4.3 Working Mechanism

Converting Reading Units to Lateral Deviations

Readings displayed by a Digitilt indicator are proportional to the angle of tilt (θ). With a metric probe, the indicator displays readings as sinθ x 25000 With an English probe, the readings are displayed as sinθ x2000

Normally, reading units are converted to lateral deviation by computer software, such as the DMM or the Digipro programmes The method that it adopts for conversion is as shown below :

Supposing that the manual reading that the indicator shows with a metric probe is as given below

Manual mode . Metric
A=321 B= -289

Divide the reading by 25000 to obtain the sinθ value for each axis.

$$\sin\theta\ (\text{A axis}) = \frac{321}{25000}$$

$$= 0\ 01284$$

$$\sin\theta \text{ (B axis) } = \frac{-298}{25000}$$

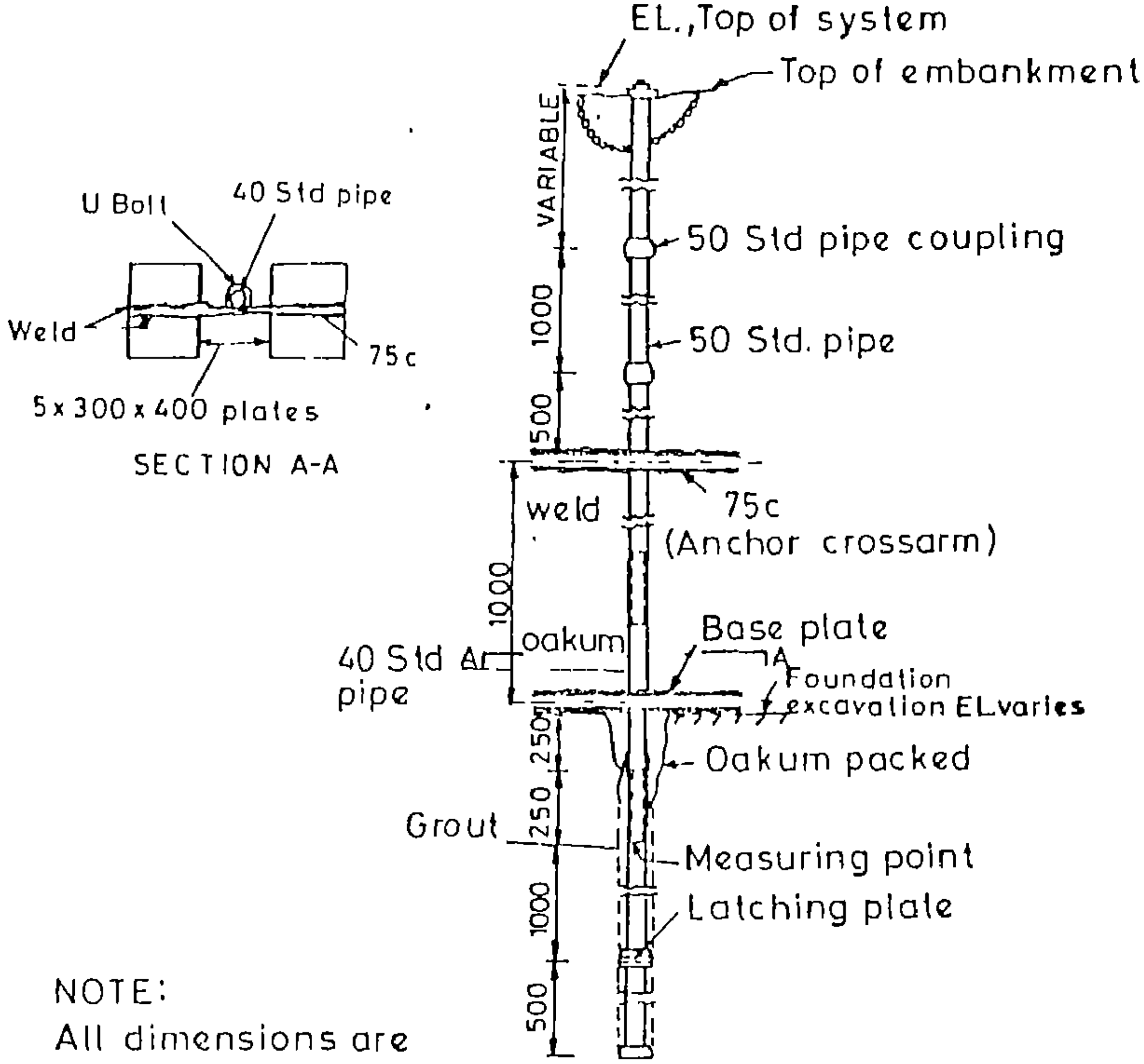
$$= -0\ 01192$$

Now multiply sinθ by the measurement interval The interval used with metric probes is typically 0 5 m or 500 mm

Lateral Deviation (A- axis) = 0 0128 x 500 mm
= 6 42

Lateral Deviation (B- axis) = -0 0119 x 500 mm
= -5 96 mm

Therefore it could be said that a lateral deviation 6 42 mm occurs in A0 dircection and -5 96 mm in B0 direction or +5.96 in B180 direction



NOTE:
All dimensions are
in mm
If necessary dimensions
can be changed

FIG.3.1 SETTLEMENT PLATE INSTALLATION

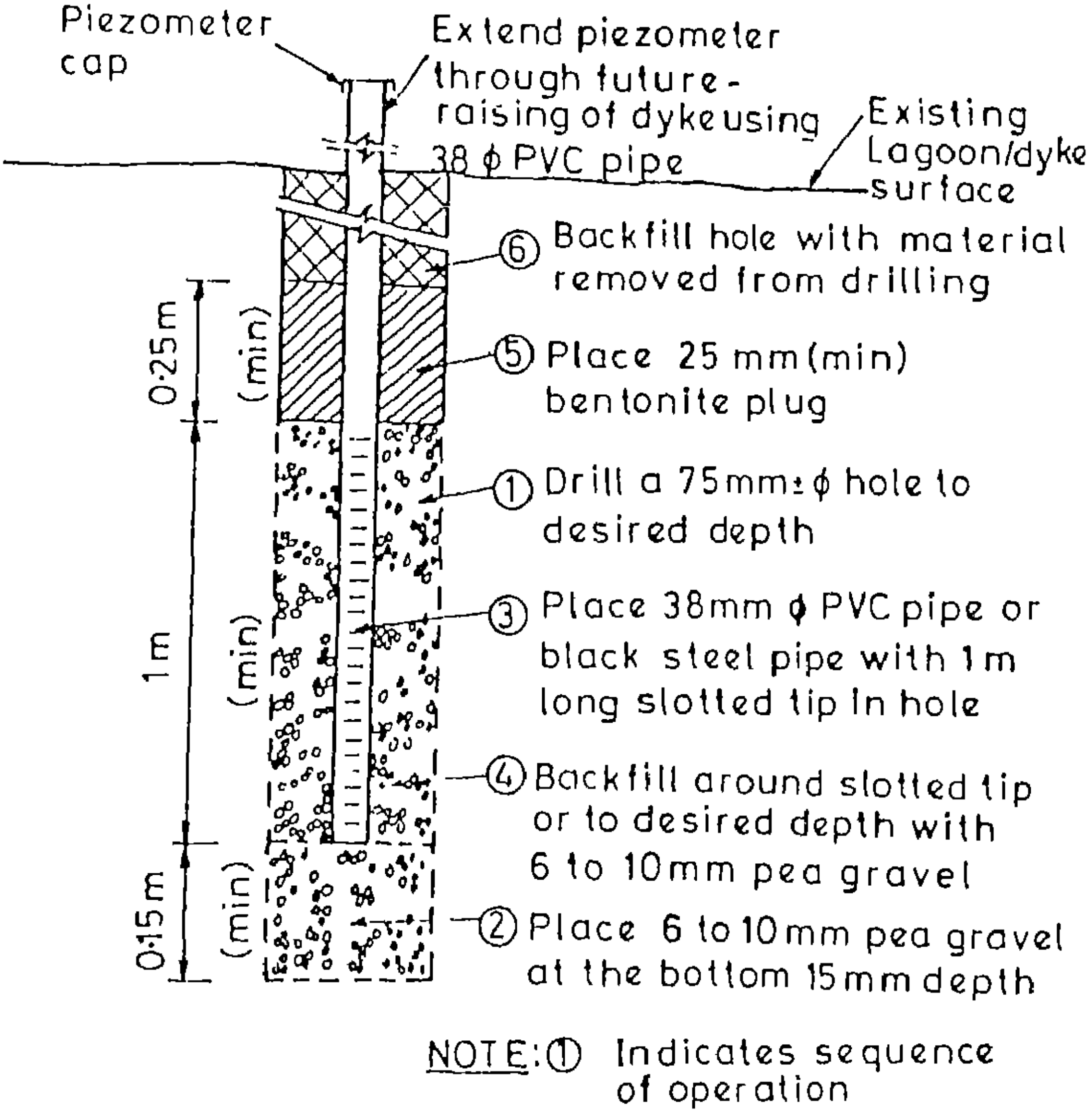

Piezometer cap
Extend piezometer through future-raising of dyke using 38 ϕ PVC pipe
Existing Lagoon/dyke surface
⑥ Backfill hole with material removed from drilling
⑤ Place 25 mm(min) bentonite plug
① Drill a 75mm±ϕ hole to desired depth
③ Place 38mm ϕ PVC pipe or black steel pipe with 1m long slotted tip in hole
④ Backfill around slotted tip or to desired depth with 6 to 10mm pea gravel
② Place 6 to 10mm pea gravel at the bottom 15mm depth
0·25m (min)
1m (min)
0·15m (min)
NOTE: ① Indicates sequence of operation

FIG.3.2 NEW PIEZOMETER INSTALLATION

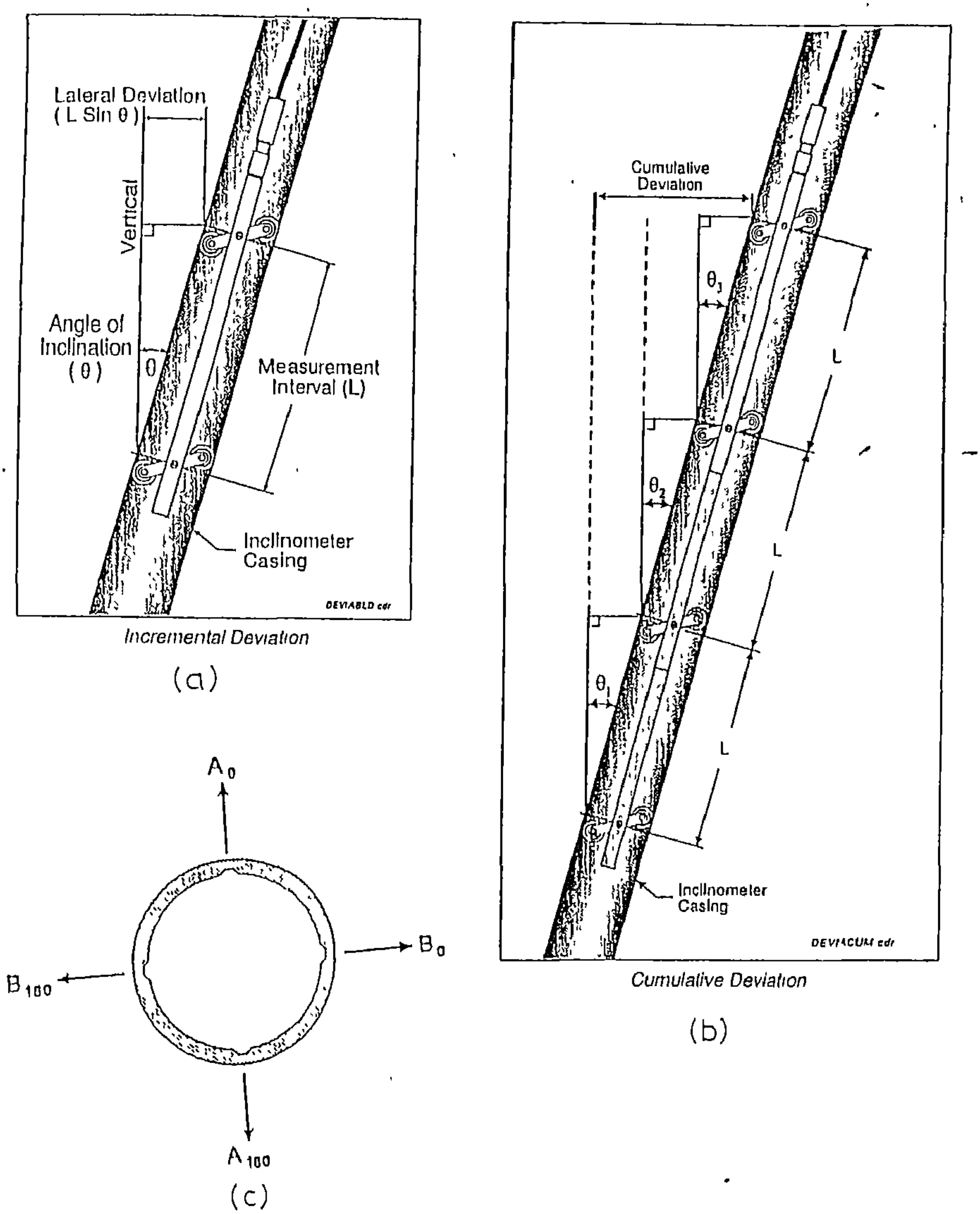


Fig. 3.3 Inclinometer details

Chapter 4

Testing Programme

4.1 General

Two sites were chosen for the post construction monitoring of ash dykes, one at Korba Super Thermal Power Plant (KSTPP), Korba in Bilaspur District and the other at Badarpur Thermal Power Station (BTPS), New Delhi. Necessary field and laboratory tests were conducted at both of these sites. Standard Penetration Tests were performed at KSTPP and BTPS ash dyke sites Piezocone tests were also conducted at BTPS ash dyke site. Existing correlation between field tests and geotechnical parameters were used to evaluate the shear strength properties of ash. To measure the horizontal and vertical movements within the ash embankment constructed over the lagoon ash, settlement plate and inclinometers were installed at KSTPP ash dyke, in addition to piezometers

4.2 Site Selection

The ash dyke site of Korba Super Thermal Power Plant **(KSTPP)**, Korba was chosen as it is the first detailed designed ash pond which is raised by *upstream method* of construction utilizing ash as a main dyke constructional material The scheme has been implemented over a decade ago and since then four stages of effective height 3 m each have been raised. This is only site in India where so many stages have already been constructed Thus this site has a tremendous importance as it not only utilizes its ash for constructing further raisings but also gives a new direction to the nation's other Therma Power Plants. Thus monitoring of ash dyke at KSTPP was a must This site would provide a good check over the assumption of design parameters and functioning of the ash dyke and eventually may lead to some modifications in the design assumptions, if found necessary during the course of monitoring

The other ash dyke chosen was Badarpur Thermal Power Station **(BTPS)**, New Delhi, as with many other sites in India, this site has also undergone only one raising so far and additional raising shall be conducted in the near future. Since this site is located in Delhi, it is very convenient from logistic point of view and cumbersome field testing such as Piezocone test could easily be performed At this site, there's a strict check over the pollution caused by the disposal of ash either in air or water, therefore it provides an additional impetus from environmental point of view. Many of the drawbacks of KSTPP ash dyke such as the problem of fugitive ash, high concentration of ash in final discharge of water into river etc were minimized at BTPS ash dyke. Another important consideration was that BTPS ash dyke lies in seismic zone IV which is an earth-quake prone area as compared to KSTPP site which lies in zone I.

4.3 Site Description

Description of the existing conditions at the selected sites of KSTPP and BTPS ash dyke are summarized here The summary of design details of KSTPP ash dyke has also been presented in this section Since the design of BTPS ash dyke is similar to that of KSTPP ash dyke, therefore the design details of the former has been omitted

4.3.1 Korba Super Thermal Power Plant (KSTPP)

The existing ash disposal site is located near Korba Super Thermal Power Plant, Korba, in Bilaspur District of Madhya Pradesh The Korba Super Thermal Power Project has an approved ultimate installation capacity of 2100 MW The coal source and water source of the power plant are Kusmunda Block Gevra Mines and Hasdeo River respectively The ash production for each 200 MW and 500 MW units are 60 T/hr and 150 T/hr, respectively , amounting an yearly production of 3.5 million tonnes of ash from all the six units The maximum slurry discharge is 5600 cum/hr.

The ash produced at KSTPP is mixed with water and pumped to the ash disposal site through pipe lines The ash disposal area is approx. 750 acres with earthen dykes all around to form a lagoon. Divide bunds across the dyke separates the lagoon into three parts. The existing ash disposal system at KSTPP ash dyke is shown in Figure 4.1 The outer starter dyke has the crest elevation of 286 m and the divide bund has the crest elevation of 284 m., while the original ground level is at an elevation of 280 -282.5 m. Five addtional stages of an effective height of 3 m each has been proposed for the raisings of the original starter dyke. So far, three raising has already been constructed and the fourth raising is in progress, thus a crest elevation of 295 m for the third raising has already been achieved successfully. At the time of field investigation in the month of December 1996 the ash was being discharged into Lagoon II which was nearly full and the third raising of lagoon III was just completed. Hence arrangements were being made to shift the discharge from lagoon II As shown in Figure 4 1 lagoon III is very small and therefore, by the time lagoon III is half filled, lagoon I should be ready for receiving the ash slurry Ash slurry discharged into the lagoon is allowed to stagnate and the ash particles settle down at the bottom leaving almost a clear water at the top. This clear water and the rain water collected within the pond is drawn out through tower type decanting water escape structures provided within the lagoons

4.3.1.1 Summary of Ash Disposal Facility Design

The ash refuse disposal plan is provided for the disposal of ash produced at KSTPP for a period of 25 years. The design was based on the field investigation and laboratory testing of KSTPP ash, *in situ* soil and borrow material. In all cases the dyke has been constructed by upstream method on the settled ash bed, using ash as the main constructional material. The cross- section of complete ash embankment is shown in Figure 4.2 and cross-sectional details of ash dyke in Figure 4.3 . The ash disposal facility design includes

Stage I This existing stage includes an outer dyke constructed to El 286 0 m using earth material compacted to designed density Divide bunds, constructed to El 284.0 m using earth material, separates the lagoon in three parts viz Lagoon I, II and III The outer dyke consists of internal drainage system and rock drain The divide bund has no internal drainage system

Stage II · This Stage includes the raising of the divide bund to crest El 289 0 m by upstream method of construction. The ash embankment has been compacted to required density and consists of internal drainage system. The entire ash embankment has been then covered with soil cover and the finished surface has been vegetated Each stage of ash dyke consists of internal drainage system and rock toe-drain.

Stage II through Stage VI : Stages II through VI would be continued to raise the dyke crest by upstream construction method using lagoon ash in a controlled fashion above settled lagoon ash placed during previous stages At present there has been a total raising upto stage IV i e. three raisings has already been done using pond ash as a main construction material Each stage of embankment crest has an effective height of 3 m In stages II, III, IV, V and VI embankment crest is raised to El. 289 0 m, El 292.0 m, El. 295 0 m, El 298.0 m and El 301 0 m, respectively

Internal Drainage System : A continuos internal drainage system has been installed for the raising of outer dyke to collect seepage water through the body of entire embankment A drainage blanket of 0.5 m thickness has been laid at a depth of 3.5 m from the crest of each embankment The seepage water collected through the drainage blanket is carried away by perforated PVC pipe of a minimum diameter of 75 mm provide along the toe of ash dyke at 30 m c/c Finally the discharge is carried out through a system of precast concrete channel beyond the existing dyke Chimney drains are not provided in divide bunds, since divide bunds does not require a continuos internal drainage system, as each previous stage will be buried completely by deposition of ash on both upstream and downstream sides.

There are five water escape structures called *'wells'*, one in lagoon I and two in each of the lagoon II and III. These filtered water thus collected through these wells are then taken to the filtering unit from where it is discharged into river. The limit of 100 ppm of ash effluent is successfully achieved through this type of arrangement.

4.3.2 Badarpur Thermal Power Station (BTPS)

Badarpur Thermal Power Station (BTPS) is located at New Delhi-Haryana border and has a installed capacity of 720 MW The BTPS Ash Disposal site is on the western bank of river yamuna It occupies an area of approximately 885 acres. The key plan of the ash disposal site is as shown in the Figure 4 4. Out of total five phases, phase I through IV have already been constructed and the construction of phase V is in progress The original ground level is approximately at an elevation of 198 0 m. The starter dam (stage I) was built to El 206 0 m using earthen material for all phases. Three phases namely I, II and III have already been raised to stage II (El 209.0 m), using pond ash as main construction material and are already filled up with the settled ash Phase IV is currently being used Water Bound Macadam (WBM) roads have been constructed on the crest of the embankment which besides providing a convenient access road also partly saves the embankment from erosion.

The original cross-section of the earthen embankment and the ash embankment raisings has been designed very similarly to that of Korba To minimize the problem of fugitive ash, the entire exposed inactive area along with the slopes of embankment have been covered with plants and vegetation The active pond is also partially covered with *water-hyacinths* (*Jal-Kumbhi*) and the exposed settled ash surface is always kept submerged under water There is a networking of pipes over the dyke which carry sludge to facilitate plant growth The ash slurry is discharged in a garlanding fashion that provides an uniform distribution of coarser and finer particles of ash over the entire pond area The water escape structure is surrounded by *water-hyacinths* to facilitate sedimentation and reduction of sediment load in the discharging water . The *water-hyacinths* are found to be useful in trapping the finer particles of ash. It was observed that water which is finally discharged through stream into river Yamuna by the decanting structure, is relatively clear

4.4 Field Testing Programme

4.4.1 KSTPP Site

4.4.1.1 Standard Penetration Test (SPT)

Five Standard Penetration Tests were performed at KSTPP ash dyke, one each on the crest of stage I (starter dyke), stage II, stage III and stage IV of the existing raisings and one in the ash pond at locations shown in Figure 4.1. SPT were conducted at a depth interval of one meter and continued till the soil stratum beneath the dyke was encountered Tests were planned to cover the entire alignment of all the four stages of the embankment. The plot of this SPT (N-values) with depth of ash dyke at the test locations are shown in Figure 4.5 along with the cross-section inferred from the soil/ash samples collected during the test. The N-values are tabulated for values in Table 4.1. It also shows the corrected N-values (in parenthesis) applying the correction factor for overburden as discussed in Figure 2 9 .

Table 4.1 SPT Results at KSTPP Ash Dyke (observed)

Elevation (m)	Settled Ash	IIIrd Raising	IInd Raising	Ist Raising	Starter Dyke
295.0	2(3)	13(21)			
294.0	4(6)	17(24)			
293.0	8(12)	20(25)			
292.0	2(3)	10(12)	9(14)		
291.0	2(3)	10(12)	8911)		
290.0	3(4)	11(13)	9(11)		
289.0	4(5)	12(14)	24(23)	7(11)	
288.0	6(8)	19(18)	14(16)	8(11)	15(24)
287.0	7(9)	17(17)	27(24)	10(13)	18(24)
286.0	20(20)	21(19)	24(21)	22(21)	20(24)

* The corrected N- values are given in the parenthesis

Results

It is observed from the table that the corrected N- values in settled ash of the pond is in the range of 3-9 except one which is 12 Most of the values of the top few metres in settled ash are in the range of 3 to 4 This remarkabe improvement in the N-values of the settled ash beneath the compacted embankment may be attributed to the consolidation, compaction and aging process. As it can be seen from the Table 4.1, the N-values of the settled ash underlying the compacted ash embankments are well above the N-values observed for the lagoon ash. The N-values of settled ash within the embankment are above 10

Figure 4.5 shows a generalised cross-section of the embankment stagings, the entire cross-section of embankment has been divided into several layers of

Table 4.2 : Angle of internal friction 'ϕ' values obtained from N-values at KSTPP ash dyke

Settled Ash

Depth	Material	N_{avg}	ϕ
0 - 8 m	settled ash	5	28°
9	clayey soil	20	33°

IIIrd Raising

Depth	Material	N_{avg}	ϕ
0 - 3.5 m	compacted ash	23	35°
3 5 - 6.5 m	settled ash	13	31°
6.5 - 9 0 m	settled ash	18	32°
10.0	clayey soil	19	32°

IInd Raising

Depth	Material	N_{avg}	ϕ
0 - 3 5 m	compacted ash	12	31°
3 5 - 9.0 m	settled ash	21	33°
10.0 m	clayey soil	21	33°

Ist Raising

Depth	Material	N_{avg}	ϕ
0 -3 m	compacted ash	12	31°
4m	clayey soil	21	33°

Starter Dyke (Earthen Embankment)

Depth	Material	N_{avg}	ϕ
0 -3 m	clayey soil	24	35°

Table 4.3 Geotechnical Properties of Soil/Ash Materials

Material type	Total unit weight kg/m^3 (pcf)	Saturated unit weight kg/m^3 (pcf)	Cohesion C′ kg/m^2 (pcf)	Angle of internal friction (ϕ) degree
Foundation soil	1750 (107)	1832 (112)	0 (0)	33 0
Soil Embankment	1750 (107)	1832 (1120	0 (0)	33 0
Compacted ash	1430 (87)	1472 (90)	0 (0)	30 0
Lagoon ash	1308 (80)	1390 (85)	0 (0)	28.0
Cover Soil	1423 (87)	1472 (90)	0 (0)	30.0

settled ash layers such that each layer depicted an average N- value and corresponding angle of internal friction 'ϕ' was chosen from the chart given in Figure 2 9 Thus each layer of fly ash beneath the embankment depicted a certain value of 'ϕ'. These improved values of 'ϕ' are shown in the Table 4 2

The geotechnical parameters initially used while designing the ash dyke is shown in Table 4.3 .The angle of internal friction of lagoon ash was taken as 28° and that of compacted ash as 30°. But SPT results revealed a marked improvement in the values of 'ϕ' of settled ash below the embankment

The improvement of 'ϕ' from 28° to 30° of the settled ash beneath the compacted ash embankment is due to any of these or combination of the following factors:

(a) consolidation and subsequent improvement of othe strength of settled ash due to aging

(b) consolidation and subsequent improvement of the strength of the settled ash due to embankment loading and

(c) compaction of settled ash due to vibrocompactor and compaction equipments used for the construction of compacted ash embankment.

For the compacted ash the 'ϕ' value is found between 31° to 35°. These values are in good agreement with design value of 30° used initially for ash embankment design.

4.4.1.2 Settlement Plate

Cross arm settlement plates were installed in each of the raisings of ash dykes at KSTPP and adjoining Balco Captive Power Plant (BCPP), operated by NTPC to measure the settlement of the lagoon ash below the compacted zone of the ash embankment. However, the settlement plate installed at KSTPP ash dyke site is presently non operational due to malfunctioning. The settlement plate at BCPP ash dyke is functional and therefore the record of this settlement plate was analyzed. It is worthwhile to mention here that BCPP ash dyke is designed on similar basis, using same geotechnical parameters of ash as that of KSTPP ash dyke

Settlement plates at BCPP as well as KSTP ash dykes have been installed near water escape structure on each stage of raised ash embankment The depth of the compacted zone of the embankment is 4 m. Unfortunately settlement records of first 14 months of installation date were not available However, record of one of the settlement plates installed in stage I of lagoon II at BCPP ash dyke is shown in Table 4 4.

Results

The readings of settlement plates are taken at a interval of one month. The record indicates that there has been a total settlement of 60 mm over a period of 6 months of the lagoon ash under first raised embankment, after which the settlement has seized to occur. The settlement measured at KSTPP ash dyke of the settled ash underneath the first raising was 10 mm in 4 months after which no settlement could be detected.

Table 4.4 Settlement Plate Record

DATE	Top R.L. of Dyke (m)	Settlement (mm)
15/3/96	294.000	0
13/4/96	293 990	10
23/5/96	293.980	20
24/6/96	293.965	35
30/7/96	293.950	50
26/8/96	293 945	55
21/9/96	293.940	60
15/10/96	293.940	60
15/11/96	293.940	60
28/12/96	293.940	60
30/1/97	293.940	60

4.4.1.3 Inclinometer

Inclinometer was installed at KSTPP ash dyke in the III rd raising of the lagoon III slightly at downstream side of embankment on 11th December 1996 At the time of installation Lagoon II was almost filled up to stage IV and lagoon III was just ready to receive new ash -slurry discharge. It was therefore, considered to be the most appropriate time for the installation of inclinometer to study the performance of embankment during complete filling opreration. The total depth of ash beneath the crest of the IIIrd raisng was 10.5 m. The installation detail and orientation of inclinometer casing has been shown in Figure 4.7. The set of readings were taken on 9th February and 23th March By this time this lagoon was almost full to a maximum allowable elevation of 294.0 m ±

Installation Procedure

Bore hole of diameter 6 inches was drilled to accomodate 3.5 inch diameter inclinometer casing Bentonite slurry was used to retain the side walls of bore hole. The depth of the bore hole was kept sufficient enough to give fixity to the inclinometer casing at the base i.e. the original ground level The lower end of casing was plugged with the casing cap. Casing segments, each of length 1.5 m, were rivetted and water proof tapes were applied over the joints so as to avoid the entry of water inside the casing. After lowering the casing into the bore hole the area between the casing and the bore hole was filled up with cement sand mix. The cement sand ratio was kept as 1:5 in lower 1.0 m while 1.10 in the rest. Cement sand ratio should be such it should neither be too lean so as to permit inclinometer casing to move within the borehole nor it should be too strong to refuse any lateral movement, if any.

Results

The results are plotted on Figures 4.8 to 4 11 . As discussed in chapter 3, the results are plotted using software DIGIPRO, in the following forms for biaxial measurements.

(a) Comparison of later readings of cumulative deviation and displacements for A and B directions of casing with initial data of 11th Dec -96 with [Figure 4 8 (a) and (b)]

(b) Comparison of readings of 23rd March 97 with that of 9th February -97 for A and B directions of casing [Figure 4.9 (a) and (b)]

(c) Record of incremental deviation and displacement for A and B directions of casing [Figure 4.10 (a) and(b)] during period of filling operation in lagoon III

(d) Record of incremental deviation and displacement for A and B directions of casing [Figure 4.11] after filling operation

From the graph of cumulative displacement and cumulative deviation it is clear that the displacement has taken place at the initial phase of ash-filling operation in lagoon III Comparing the profile of casing of 9th February 97 to that of the profile of casing when it was installed i e., 11 December 96 [refer Fig. 4.8 (a)], it is observed that a maximum displacement of 11 mm in A0 direction and -15 mm in B0 direction (or 15 mm in B180 direction, refer Fig 4 7) of the casing has occurred in a zone of 2 to 3 m depth from the crest of the embankment The orientation of casing with respect to the embankment is such that resultant of A0 and B180 points towards the downstream slope side of the embankment. Thus a resultant of 18.6 mm has occurred in the above specified zone towards the downstream side within a period of 2 months Obviously, this movement is after the installation of inclinometer. Additional movement could have occurred prior to the installation of inclinometer. However this may not be significant from the design point of view as all these movements could have been associated with the construction activity.

The dataset collected on 23th March 97 does not reveal an appreciable movement as compared to the previous data set of 9th February 97. The graph of cumulative displacement hardly exhibits any significant displacement either in A0 or B0 direction. A total movement of 0.95 mm have been recorded in A0 direction at a depth of 1 to 4 m from the crest of the embankment while 0.4 mm in B180 direction at a depth just below the compacted zone of embankment. This may be attributed to the fact that the gradual increase in the pressure on the upstream face of embankment due to increasing height of ash slurry in the lagoon III has caused all the movement to take place

On comparing the graph of incremental displacement [Fig 4 10] of 9 Feb. 97 to that of 11 Dec 96 i e. during filling operation, it is found that there are two shear zones where maximum change in the tilt of the casing has taken places The graph of incremental displacement for A0 direction [Fig. 4 10 (a)] shows that there are shear zones at a depth of 3 25 m and 10 5 m from the crest of the embankment while the graph of incremental displacement for B0 direction [Fig. 4.10 (b)] shows shear zones at 2 m and 10 5 m from the crest of the embankment Thus the maximum change in tilt has occurred at a depth of 2 to 3 m and 10 5 m. Thus it could be concluded that failure may occur at these depths The shear zone depicted at both these depths signifies a slippage at the interface of Stage III and Stage IV construction and at the base of the ash dyke.

As shown in Figure 4.11, very insignificant shear zones are observed between 9 February 97 and 23 March 97 data. On comparing the datasets of incremental deviation for detecting shear zones, if any, after the filling operation, it is found that a shear zone exist at a depth of 6 to 9 m from the crest of the embankment, but the magnitude of movement is negligible as the dataset of 23 March 97 reveals marginal change in incremental displacement of 0 20 mm (max.) as compared to that dataset of 9 Feb 97. Thus it could be concluded that all these movements are restricted only during initial stages of ash filling operation in lagoon III, after which movements seize to occur.

4.4.1.4 Piezometers

Series of piezometers were installed on the slope of each embankments upto the depth of the drainage blanket Piezometers are used to check the fuctionality of drainage blanket. Piezometers should not show the water level if drainage blanket along with the vertical chimney is functioning properly

Piezometer readings at KSTPP ash dyke revealed that the drainage blankets were working perfectly

4.4.2 BTPS Ash Dyke

4.4.2.1 Standard Penetration Testing (SPT)

The Standard Penetration Tests were performed in the phase III of BTPS ash dyke at locations shown in Fig 4.4 The tests were conducted in nearly every stage of ash dyke. At this site, SPT was conducted by Soil Engineering Consultants, New Delhi in our supervision The test locations were selected in such a way so as to give almost linear alignment, covering the entire cross-section of ash pond. SPT was also performed in an active pond on a freshly deposited ash. Undisturbed samples (UDS) were collectd at every 2 or 3 m of the depth of bore-hole.

Results

The results of SPT values obtained at BTPS ash dyke are shown in Table 4.5 and are plotted in Figure 4.12. The N-values in the northern raised embankment are comparatively higher than that obtained in the southern raised embankment Since the ash-slurry disharge location for the phase III pond were located on the northern side of pond, therefore, the coarser fraction of ash, which is relatively heavier, settled at the northern end while the finer particles travelled towards the water escape structures located at the southern end and got settled there. So conducting SPT on both the discharge end as well as water escape end of ash pond of phase III revealed that there is a differnce in the gradation of ash particles and hence in the strength parameter from the discharge end to the water escape end. The ground water was measured at a depth of 10.5 m from the pond surface.

On comparing the N-values of the test locations within the pond to that of the ash beneath the raised embankment, it is found that for northern end average N-values for the lagoon ash is 4 while it is 8 beneath the raised embankment, which decreases to 6 in the deeper zone. The N-value of the lagoon ash towards

Table 4.5 : Observed N- values at BTPS Ash-Dyke

Elevation (m)	Starter Dyke	Ist Raising	Inactive Ash Pond			Ist Raising	Active Pond
	Bore Hole #1 (north)	Bore Hole #2 (north)	Bore Hole # 3 (north)	Bore Hole #4 (centre)	Bore Hole #5 (south)	Bore Hole #6 (south)	Bore Hole #7 (Phase IV)
209		10(15)	1(2)	2(3)	3(5)	6(10)	
208		-	-	-	2(3)	-	
207		-	2(3)	2(3)	-	7(9)	
206	18 (28)*	10(13)	-	-	4(5)	-	
205	21(29)	19(23)	6(7)	4(5)	2(3)	6(7)	
204	17(22)	-	-	-	-	-	1(1)
203	11(13)	8(9)	4(5)	5(5)	3(3)	4(4)	2(2)
202	24(26)	6(7)	-	-	-	-	1(1)
201	10(10)	-	2(2)	1(1)	1(1)	1(1)	1(1)
200	-	5(5)	-	-	-	3(3)	2(2)
199	-	11	10(10)	22(19)	28(28)	36(33)	13(13)

*The values given in paranthesis are corrected N-values

Table 4.6 Angle of internal friction 'ϕ' values obtained from SPT at BTPS ash dyke.

Bore Hole -1

Zone	Material	N-value	ϕ
0-6 m	silty sand	22	33°

Bore Hole -2

Zone	Material	N- value	ϕ
0 - 5 m	compacted ash	17	33°
5 - 8 m	settled ash	8	30°
8 - 11 m	settled ash	6	29°

(Bore-Hole 2 of SPT corresponds to Test No 2, 8 and 9 of Piezocone)

Bore Hole -3

Zone	Material	N value	ϕ
0 - 3 m	compacted ash	2	28°
3 - 6 m	settled ash	4	28°
6 - 9 m	settled ash	6	29°
9 - 11 m	settled ash	2	28°

(Bore-Hole 3 of SPT corresponds to Test No 7 of Piezocone)

Bore Hole -4 , 5 and 6

Zone	Material	N-value	ϕ
0 - 11	settled ash	4	28°

Bore Hole -6

Zone	Material	N-value	ϕ
0 - 5 m	compacted ash	9	30°
5 - 7 5 m	settled ash	4	28.5°
7.5 - 11.5 m	settled ash	2	28°

(Bore-Hole 6 of SPT corresponds to Test No 4 and 5 of Piezocone) the southern end of the pond is 3 while it is 4 beneath the raised embankment, which reduces to 2 in the deeper zone. This indicates that at this site also, there is improvement in the shear strength at both ends of the dyke This confirms the findings of KSTPP site. However, the gain in the strength at this site is not as much as compared to KSTPP site.

The N-values along with the angle of internal friction 'ϕ' has been shown in Table 4 6 . The division of zones within the embankment is based upon the average N-values obtained from the test It is obvious from the table that the angle of internal friction 'ϕ' in the compacted zone of embankment is about 33° in the northern end while 30° in the compacted zone of embankment in the southern end.

4.4.2.2 Piezocone

Piezocone test were conducted at BTPS ash dyke with the help of Central Soil Mechanics and Research Laboratory (**CSMRL**), New Delhi. Testing was performed on the crest of the embankment and ash pond of phase III The pond was inactive and dry. The test locations along with their test numbers have been shown in Fig 4.4 .

Results

The piezocone results are given in the Figure 4.13 which includes cone resistance 'q_c' (MPa), sleeve resistance 'f_s' (MPa) and the friction ratio 'F.R '. Because of malfunctioning in the pore water pressure measuring section of piezocone, pore water pressure could not be measured Angle of internal friction ϕ was calculated from cone reistance values using the various relationships discussed in chapter 2 for different layers or zones

The angle of internal friction ϕ as found by Meyerhof, Trofimenkov and Schmertmann method is given in Table 4 7. Average q_c values have been used for evaluating the ϕ values of various zones or layers

Northern Embankment

In the northern embankment q_c values for the compacted zone of the embankment lies in the range of 75 to 80 kg/cm² and F R in the range of 1.0 -3 0 For the lagoon ash underlying the embankment, q_c values are in the range of 15 to 30 kg/cm² while F R. is approximately 1.0 .

The angle of internal friction ϕ as found from Meyerhof and Schmertmann methods in the compacted zone of the embankment is 36°. At greater depths i e. 8 to 11 m from the surface of the embankment ϕ of lagoon ash is found to vary from 31 5° to 33.8° from the above mentioned methods

Southern Embankment

In the southern embankment q_c values in the compacted zone of the embankment is 50 kg/cm² while in the lagoon ash beneath it is in the range of 10 to 20 kg/cm² The F.R in the compacted zone is in the range of 2.5 to 4 0 while in the ash beneath is 2.0 -4.0 .

The angle of internal friction ϕ in the compacted zone is found out to be 34°-35°. At greater depths i e. in the lagoon ash, ϕ is found to vary in the range of 27°- 31.8°.

Remarks

On comparison in the data sets of northern and southern embankment, one can visualize that q_c values are higher in the northern embankment than the southern This indicates that coarser particles are on the northern end of the pond. This is also justified by the friction ratio values. Lower friction values indicate coarser nature of particles which is the case in northern end. On the contrary southern end exhibits higher F.R. and lower q_c indicating finer nature of particles of fly ash. These results are qualitatively in conformance with the SPT results The variation of friction ratio with soil type has been shown in Figure 4.10 (Schmertmann ,1969)

Table 4.7 : Angle of internal friction ϕ obtained from Piezocone test

Northern Embankment (near active pond)

Test No.2

Zone	$\sigma'_{mid-depth}$	q_c	Meyerhof	Trofimenkov	Schmertmann	
(m)	(kg/cm²)	(kg/cm²)	φ	φ	D_r	φ
0 - 5	0 2756	75	36°	36°	-	-
5 - 8	0 717	30	34°	29°	40%	38-34 5
8 - 10.5	1 02	20	33°	-	30%	37-33.8

Test No. 8

Zone	$\sigma'_{mid-depth}$	q_c	Meyerhof	Trofimenkov	Schmertmann	
(m)	(kg/cm²)	(kg/cm²)	φ	φ	D_r	φ
0 - 4.5	0.298	75	36°	36°	-	-
4.5-8.5	0 72	25	33°	29°	35%	38-34
8.5-10	1 02	20	32.5°	-	25%	36-33

Test No 9

Zone	$\sigma'_{mid-depth}$	q_c	Meyerhof	Trofimenkov	Schmertmann	
(m)	(kg/cm²)	(kg/cm²)	φ	φ	D_r	φ
0 - 5	0.278	80	36°	36°	-	-
5 - 8.5	0.744	25	33°	29°	35%	38-34
8.5-10	1.075	15	31.5°	-	20%	35.5-32 8

Southern Embankment

Test No. 4

Zone	$\sigma'_{mid-depth}$	q_c	Meyerhof	Trofimenkov	Schmertmann	
(m)	(kg/cm^2)	(kg/cm^2)	φ	φ	D_r	φ
0 - 5	0.29	50	35°	34°	93 %	45-39.5
5 - 7.5	0.729	20	32°	28°	30%	37-33.7
7.5-11.5	1.109	10	-	-	10%	34.2-31.8

Test No. 5

Zone	$\sigma'_{mid-depth}$	q_c	Meyerhof	Trofimenkov	Schmertmann	
(m)	(kg/cm^2)	(kg/cm^2)	φ	φ	D_r	φ
0 - 4.5	0.25	50	35°	35°	90%	45-39
4.5-7.5	0.66	25	33°	29.5°	38%	37.5-34.5
7.5-8.5	0.88	20	32°	28°	25%	36 - 33
8.5-10.5	1.05	10	-	-	10%	33.5-31

Settled Ash
(At the end of ramp)

Test No. 7

Zone	$\sigma'_{mid-depth}$	q_c	Meyerhof	Trofimenkov	Schmertmann	
(m)	(kg/cm^2)	(kg/cm^2)	φ	φ	D_r	φ
0 -3	0.16	15	31.5°	33°	45%	37-33
3 - 6	0.487	25	33°	31°	45%	39-35
6 - 9	0.812	25	33°	29°	35%	38-34
9 - 11	1.08	25	33°	-	30%	37-33.7

(Corner Site)

Test No 6

Zone	$\sigma'_{mid-depth}$	q_c	Meyerhof	Trofimenkov	Schmertmann	
(m)	(kg/cm^2)	(kg/cm^2)	φ	φ	D_r	φ
0 -3	0.16	15	31.5°	33°	45%	37-35
3 - 10	0.7	20	32°	28°	30%	37-33.7

Settled Ash of Pond

In the inactive pond piezocone tests were performed at the northern end of the pond The test could not be perfomed at other locations due to inaccessibility of site for 18 ton truck mounted piezocone rig The piezocone results indicate that ϕ values are from 29°- 33° of the settled ash. The friction ratio is found to vary from 1.0 to 1 5, indicating herein also the accumulation of coarser material at the discharge end. The q_c values are comparable with those of the lagoon ash beneath the embankment.

4.4.2.3 Plate Load Test

The test was conducted by NTPC and the results are analysed here Angle of internal friction was found through two plate load tests that were conducted on the ash pond The water table at the site was quite low. Plate load tests were conducted in the ash pond in the test pits of size 4m x4m x1 5m. The size of the plate was 75 cm x 75cm x 2.5 cm. The width of test pit was kept slightly greater than 5 times the width of the plate The resulting plot of stress vs settlement are shown in Figure 4 14

Results

Of the two plate load tests, one exhibited a general shear failure while the other a local shear failure. The ultimate bearing stress was observed to be 0.95 kg/cm² in the case of general shear failure From the other test ultimate bearing stress was found out to be 0.7 kg/cm² from log-log graph of stress vs settlement. Angle of internal friction ϕ was indirectly calculated from the Terzaghi's bearing capacity values.

Table 4 8 ϕ values from Plate Load Test

q_u (kg/cm²)	N_γ	ϕ
0.95	19.37	32
0.7	14 27	30

Table 4.10 : Friction Ratio (Schmertmann, 1969)

Soil Type	Friction Ratio percent
Very shelly deposits lime rock (soft, shelly partially indurated limestone)	0.0 - 0.5
Clear sand, no plastic fines (independent of relative density)	0.5 - 2.0
Silty sand	1.75 - 2.5
Clayey sand, silts, marls, moderately sensitive clays	2.33 - 3.5
Sandy clay	3.00 - 4.5
Relatively insensitive clay	over 4.0

4.5 Laboratory Testing Programme

The laboratory tests were necessary to correlated the various in situ geotechnical parameters to the laboratory results. Laboratory tests were conducted mainly on BTPS ash sample collected from different depths and locations of ash pond during field testing programme. Undisturbed samples were also analyzed by Soil Engineering Consultants, New Delhi Some tests which were felt necessary and not performed earlier during the design of KSTPP ash dyke, were also conducted. The other geotechnical parameters of KSTPP ash are taken from KSTPP-design report prepared by Dayal (1988) All tests were performed according to Indian standards.

4.5.1 Grain Size Distribution

The grain size analysis was performed on the pond ash collected from BTPS ash pond and is shown in Figure 4.15 . The result for the grain size analysis of KSTPP ash has also been reported along with BTPS ash in Table 4 9 The BTPS ash is found to be coarser than that of KSTPP ash. The coefficient of uniformity, defined as the ratio of d_{60} over d_{10} of BTPS sample of ash is found to be 8.67 .

On the average,

for sands $Cu = 10$ to 20

for silts $Cu = 2$ to 4

for clays $Cu = 10$ to 100 (as classified by Jumikis 1962)

Table 4.9 Grain Size Analysis of Ash

Particle Size Distribution	KSTPP ash	BTPS ash
Gravel Size (%)	0	0
Sand Size (%)	68	78
Silt Size (%)	32	22
Clay Size (%)	0	0

BTPS ash lies in the range of silt and sand. While the value of C_c is defined as $d_{10}^2/ d_{60} \cdot d_{10}$ is found to be 2.88 which is between 1 and 3 and therefore it could be classified as well graded. Thus fly ash of BTPS could be termed as well graded silty -sand

4.5.2 Specific Gravity

Specific gravity of BTPS ash was determined in the laboratory with the help of specific gravity bottle and was found out to be 2.08 . Specific gravity of KSTPP ash as reported by Wasan (1995) is 1.76 The specific gravity obtained for BTPS and KSTPP ash samples fall well within the specific gravity range of 1.6 to 2.4 for Indian ashes (refer Table 2.2). Since BTPS ash has higher iron content as compared to KSTPP ash, therefore, the specific gravity of BTPS ash is also higher than that of KSTPP.

4.5.3 Compaction Characteristics

The maximum dry density (MDD) was determined using Standard Proctor mould and is shown in Figure 4.16. The maximum dry density (MDD) and optimum moisture content (OMC) of Badarpur ash is found to be 1.05 g/cc and 38% respectively, while the reports from Korba reveal a MDD of 1.03 g/cc and OMC of 42 %. These values are found to be in conformance with the range of Indian ashes of 0.935 to 1.78 gm/cc. This also justifies the fact that MDD increases with the increase in specific gravity of ash.

All the tests have been carried out using the same sample throughout the test by adding water at successive compaction operation. This can result in higher value of MDD when compared with the result obtained using separate portion of original sample at each point of the curve. When the same sample is used throughout the test the particles become increasingly finer by each successive compaction giving denser packing

4.5.4 Coefficient of Permeability

Permeability values of the samples can be determined in the laboratory using constant head or variable head permeability approach The coefficient of permeability as found from the falling head method for BTPS ash sample compacted at 95% of Standard Proctor Density is 8 5 x 10^{-5}cm /sec The coefficient of permeability of KSTPP ash is 1.85 x 10^{-4} cm/sec as reported by Dayal et al (1988) This is a typical range of 10^{-4} to 10^{-5} cm/sec for Indian ashes as given by Rajasekhar 1995.

4.5.5 Coefficient of consolidation

The consolidation behaviour of ash is expected to be similar to that of non -cohesive soils where in majority of settlement occurs within a very short period and therefore consolidation tests were conducted in triaxial apparatus on specimens prepared at 95 % of Std Proctor density Small strips of filter paper (about 1/2 inch wide with a gap of about 1/2 inch) were attached on the cylindrical circumference of the sample to enhance the radial drainage The sample was then covered with the rubber membrane. Back pressure of 1.5 kg/cm² and cell pressure of 1.7 kg/cm² (slightly more than the back pressure) was used to saturate the sample. To ensure complete saturation Skempton's parameter ($B = \Delta u / \Delta\sigma_3$) was checked with the help of pore water pressure measuring device After saturation, back pressure was closed and the sample was allowed to consolidate at 2.0 kg/cm² and 2.5 kg/cm² with drainage valve open The drainage valve was connected to the beurette so as to record volume change of the sample during consolidation. Change in volume of the specimen was plotted against square root of time. The initial part of this plot, to about 50% consolidation, is approximately linear and when it is extended to intersect the horizontal line, it represents sqare root time ($\sqrt{t_{100}}$) of 100 % consolidation.

The value of C_v is calculated from the following equation

$$C_v = \pi \frac{D^2}{n t_{100}}$$

D = Diameter of the sample

L = Length of sample

t_{100} = Time required for 100 % consolidation

n = Constant depending on the drainage condition

For L /D = 2

Drainage from one end n = 1

Radial and one end drainage n = 80

The C_v values obtained for BTPS ash (D=3.81, L=7 62) at 95 % of standard proctor density is found out to be 0.788 cm²/min (refer Fig. 4 17) for a confining stress of 2.5 kg/cm² and 1.088 cm²/min for a confining stress of 2.0 kg/cm². Thus C_v values found in the given range of 0.57 to 1 14 cm²/min by Capco (1990) It could be concluded that C_v values decreases with the increase in confining stress. Webb and Hughes (1987) have reported that if an automatic recording system is used for taking the consolidation readings at very small time intervals, C_v values would be in the range of 1.9 to 19 cm²/min

4.5.6 Compression Index

The compression index (C_c) values were obtained in oedometer apparatus. The C_c value for the BTPS ash sample, prepared at 95 % of standard proctor density, is found to be 0.115 (Fig. 4.18) and 0 199 for the ash slurry (Fig. 4.19). The C_c value obtained on ash slurry for KSTPP ash is found out to be 0 086 (Fig. 4.20) .

4.5.7 Shear strength

Angle of internal friction was found for BTPS ash sample from both triaxial and direct shear apparatus. The sample was first saturated and then consolidated for next 24 hours. Only drained test could be performed in triaxial apparatus as some problem emerged in pore water pressure measuring machine at later stage of laboratory testing due to which undrained tests with pore water pressure measurements could not be performed

The angle of internal friction as obtained from Triaxial test for consolidated drained condition of BTPS ash Sample, prepared at 95% std proctor density, was found out to be 36° (Fig 4 21).

Direct shear tests conducted on the BTPS ash sample prepared at 95 % std. proctor density for the consolidated drained condition, exhibited an angle of internal friction of 39° and for consolidated undrained condition, it was found out to be 37.5° (Fig. 4.22).

It has been found that the angle of internal friction obtained from a triaxial test (ϕ_{tr}) is generally 1 to 5 degrees lower than that from a plane strain test (ϕ_{ps}). As proposed by Lee (1970)

$$\phi_{tr} = 1.1\phi_{ps} \qquad (\phi_{tr} \geq 30°)$$

Test results obtained in the laboratory for triaxial and direct shear are found out to be in good agreement with the above mentioned equation

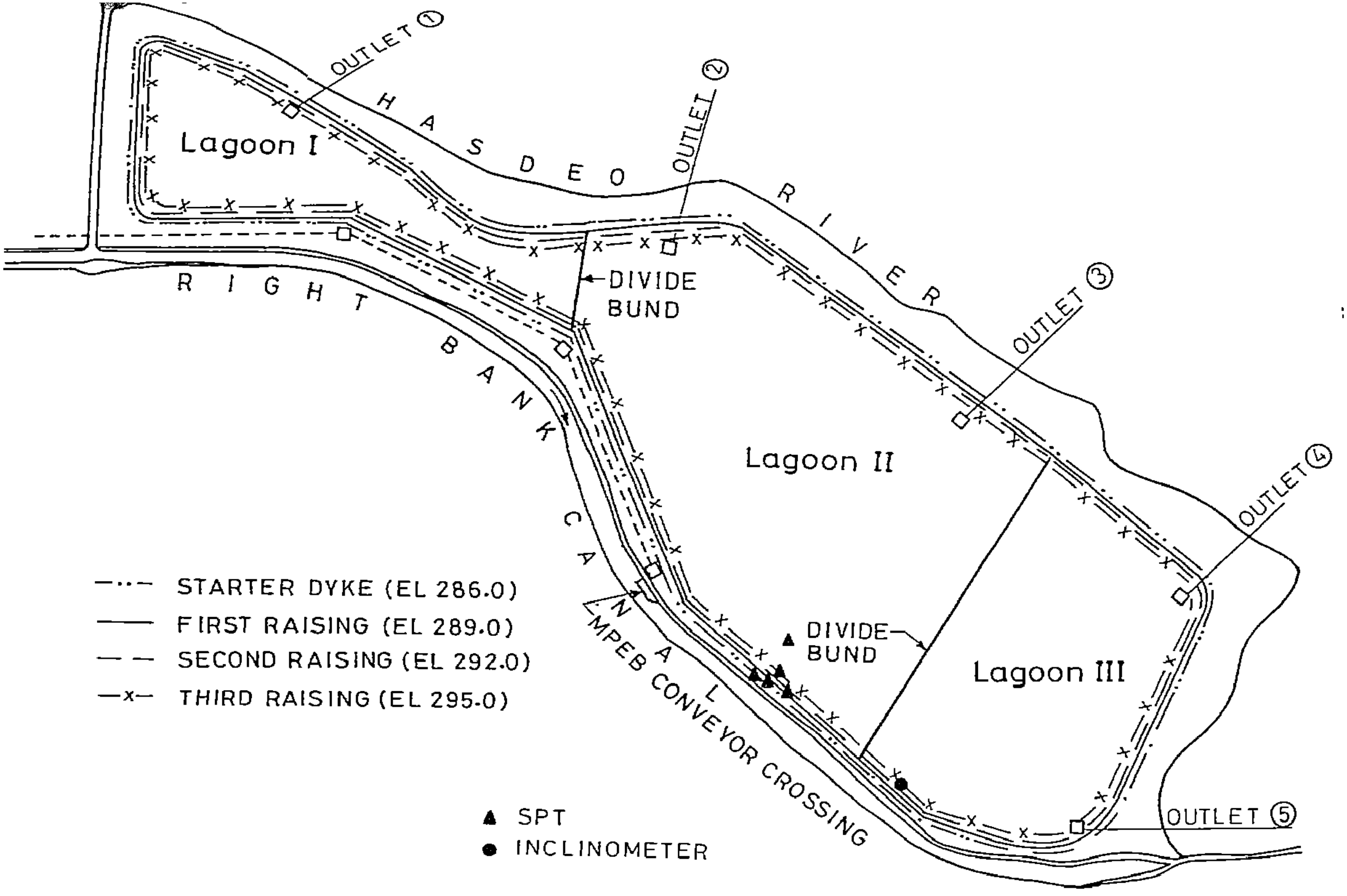


FIG.4.1 TEST LOCATIONS AT KSTPP-ASH DYKE (KEY PLAN)

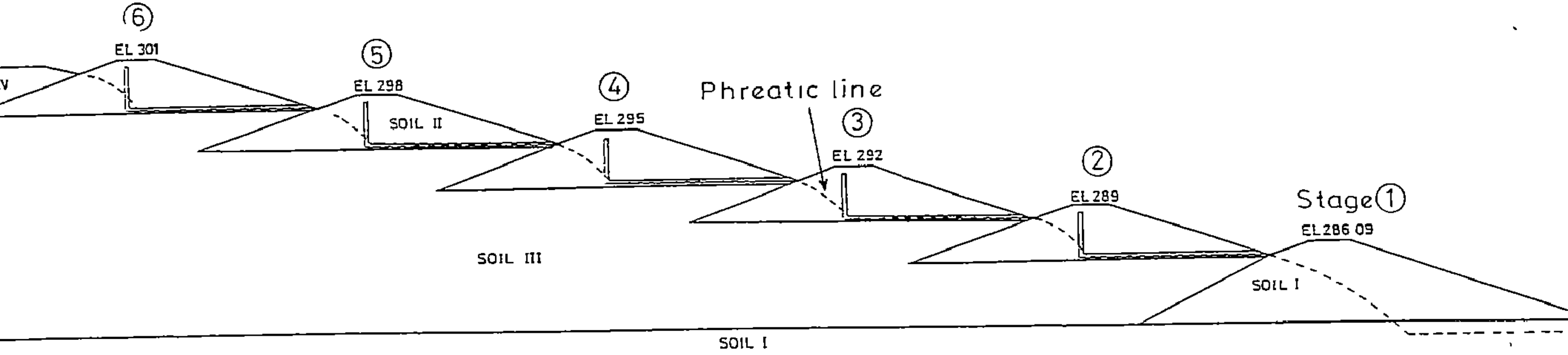


FIG. 4.2 FINAL ASH EMBANKMENT CROSS SECTION

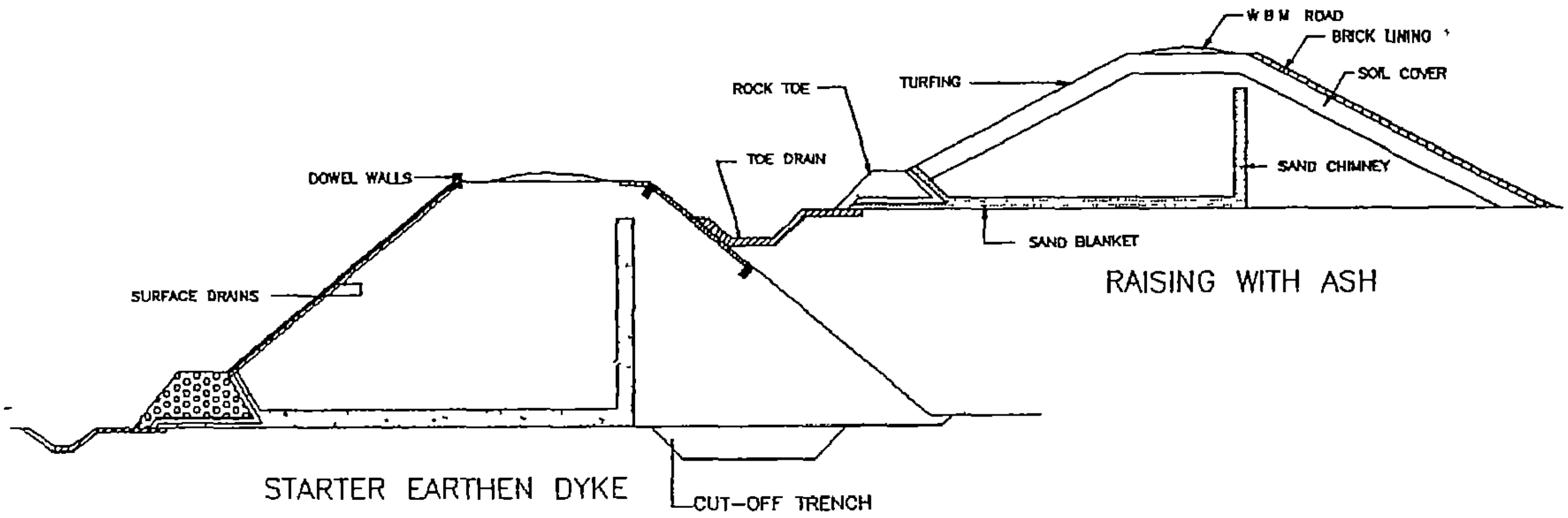
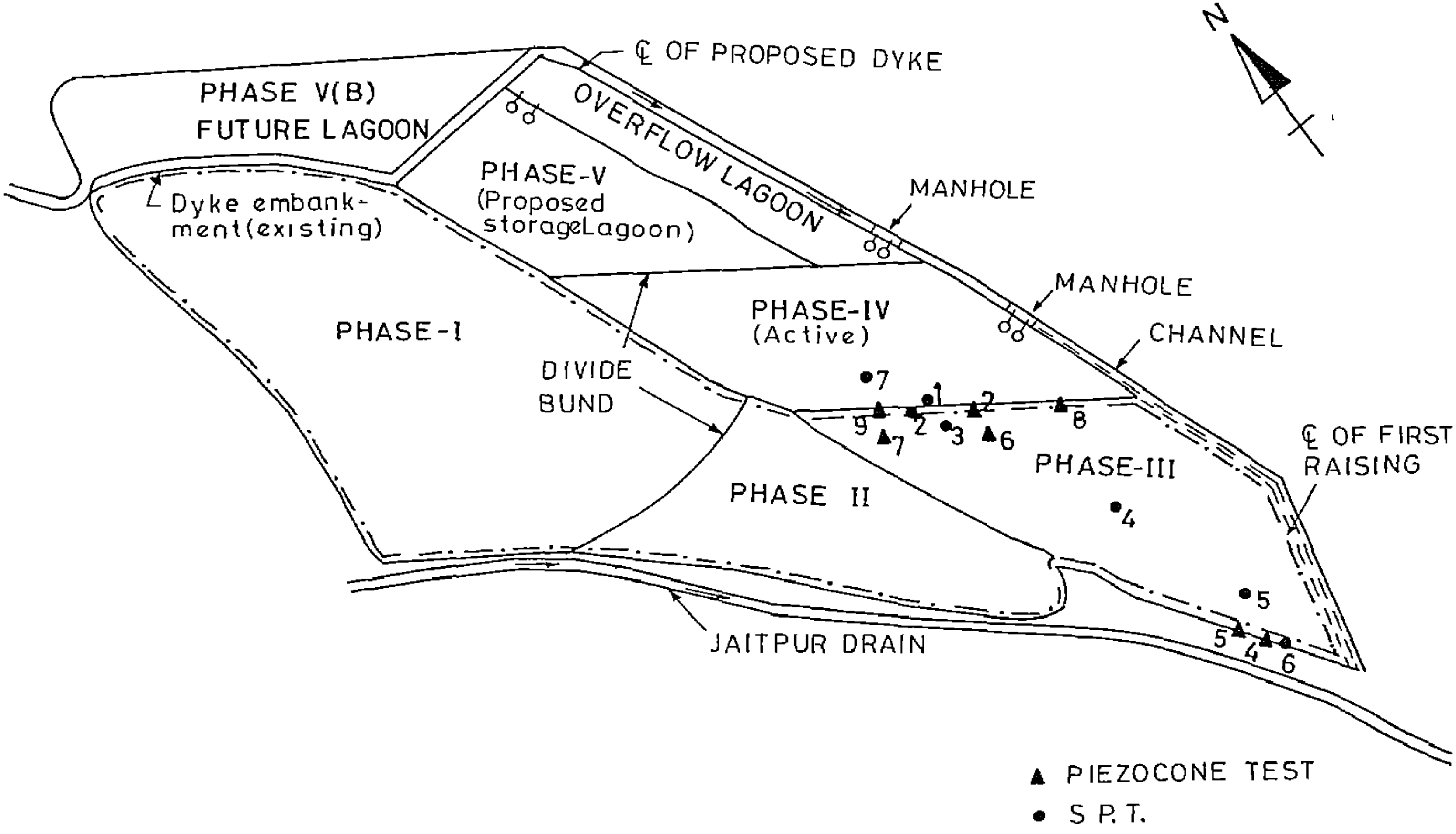


FIG.—4.3 TYPICAL SECTION OF ASH DYKE



FIG·4.4 TEST LOCATIONS AT BTPS-ASH DYKE (KEY PLAN)

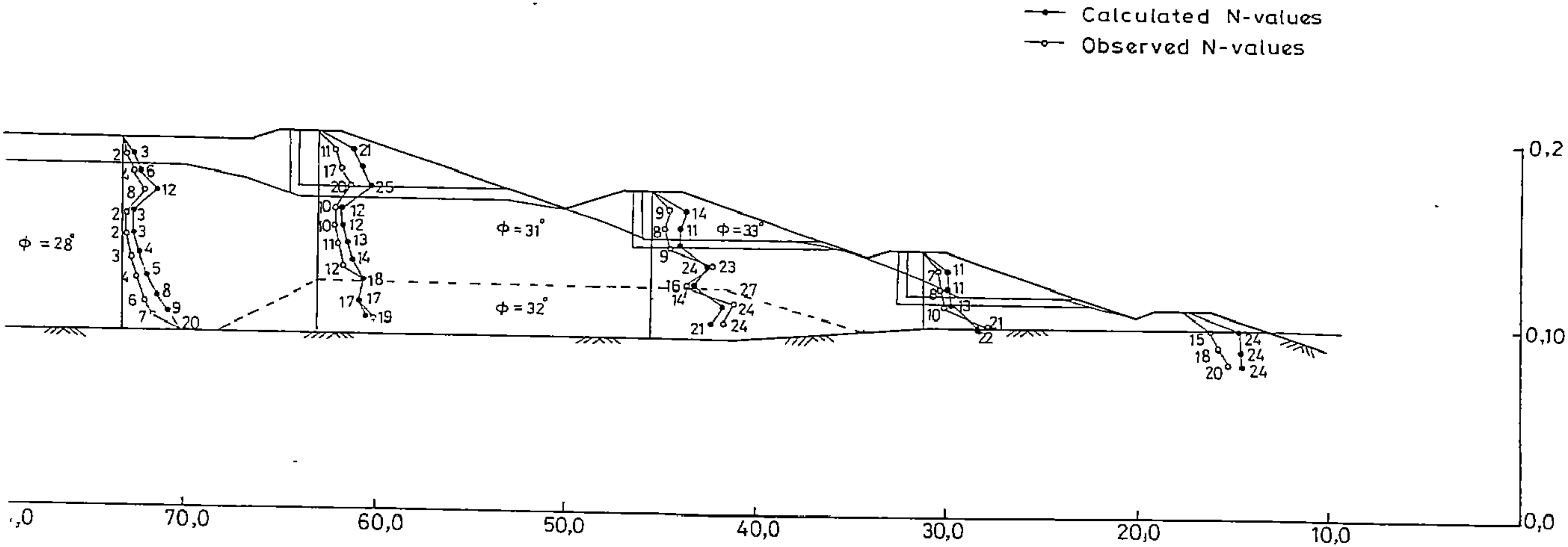


FIG 4.5 STANDARD PENETRATION TEST AT KSTPP-ASH DYKE

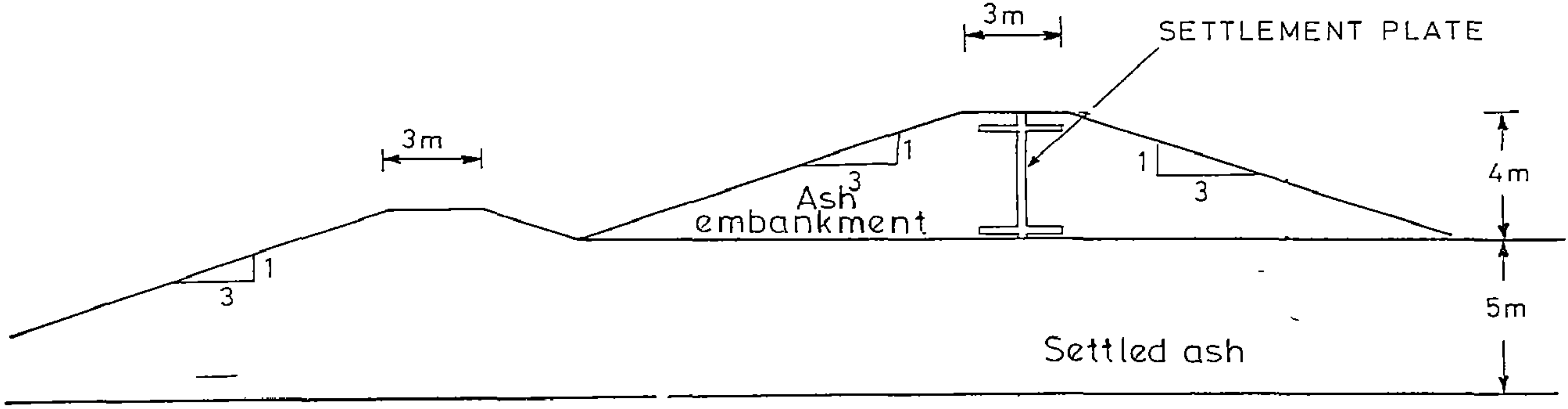


FIG. 4.6 SETTLEMENT PLATE INSTALLATION

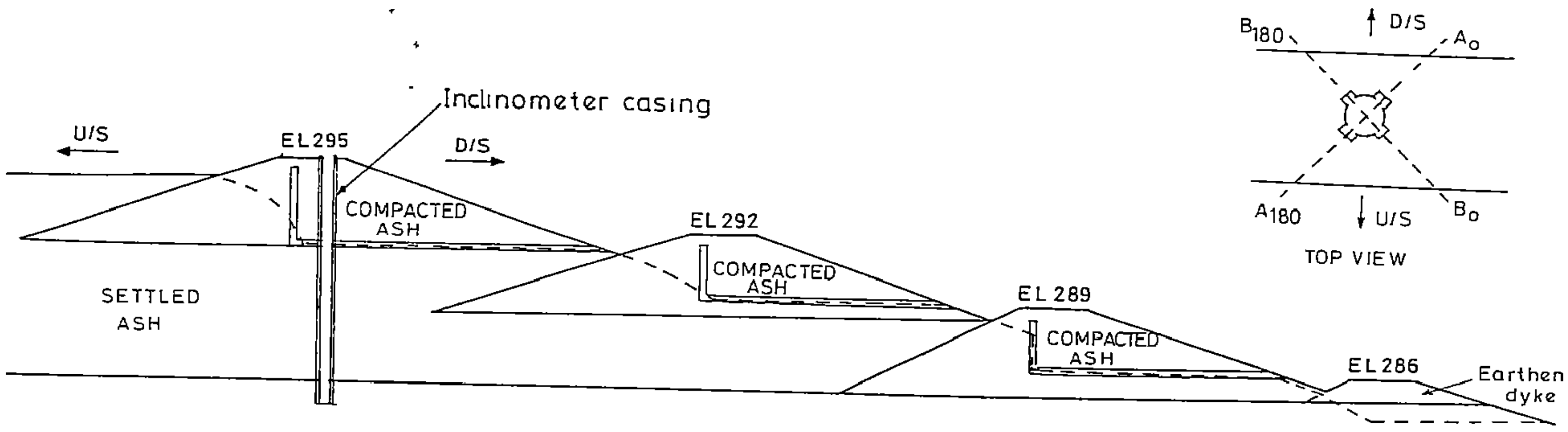


FIG 4.7 INCLINOMETER INSTALLATION AT KSTPP ASH-DYKE

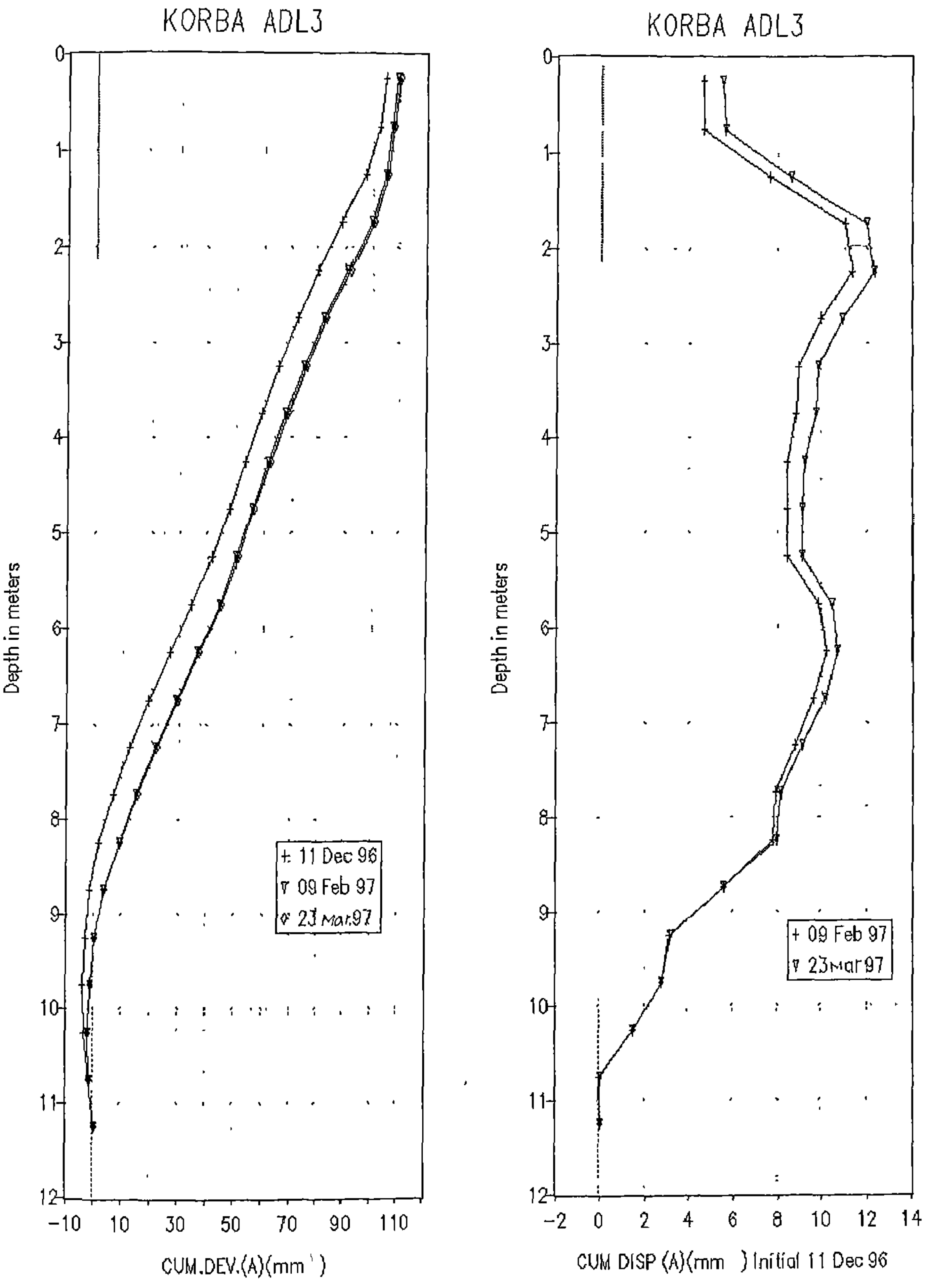

KORBA ADL3
Depth in meters
0
1
2
3
4
5
6
7
8
9
10
11
12
-10
10
30
50
70
90
110
CUM.DEV.(A)(mm)
+ 11 Dec 96
▿ 09 Feb 97
▿ 23 Mar 97
KORBA ADL3
Depth in meters
-2
0
2
4
6
8
10
12
14
CUM DISP (A)(mm) Initial 11 Dec 96
+ 09 Feb 97
▿ 23 Mar 97

Fig. 4·8 (a)

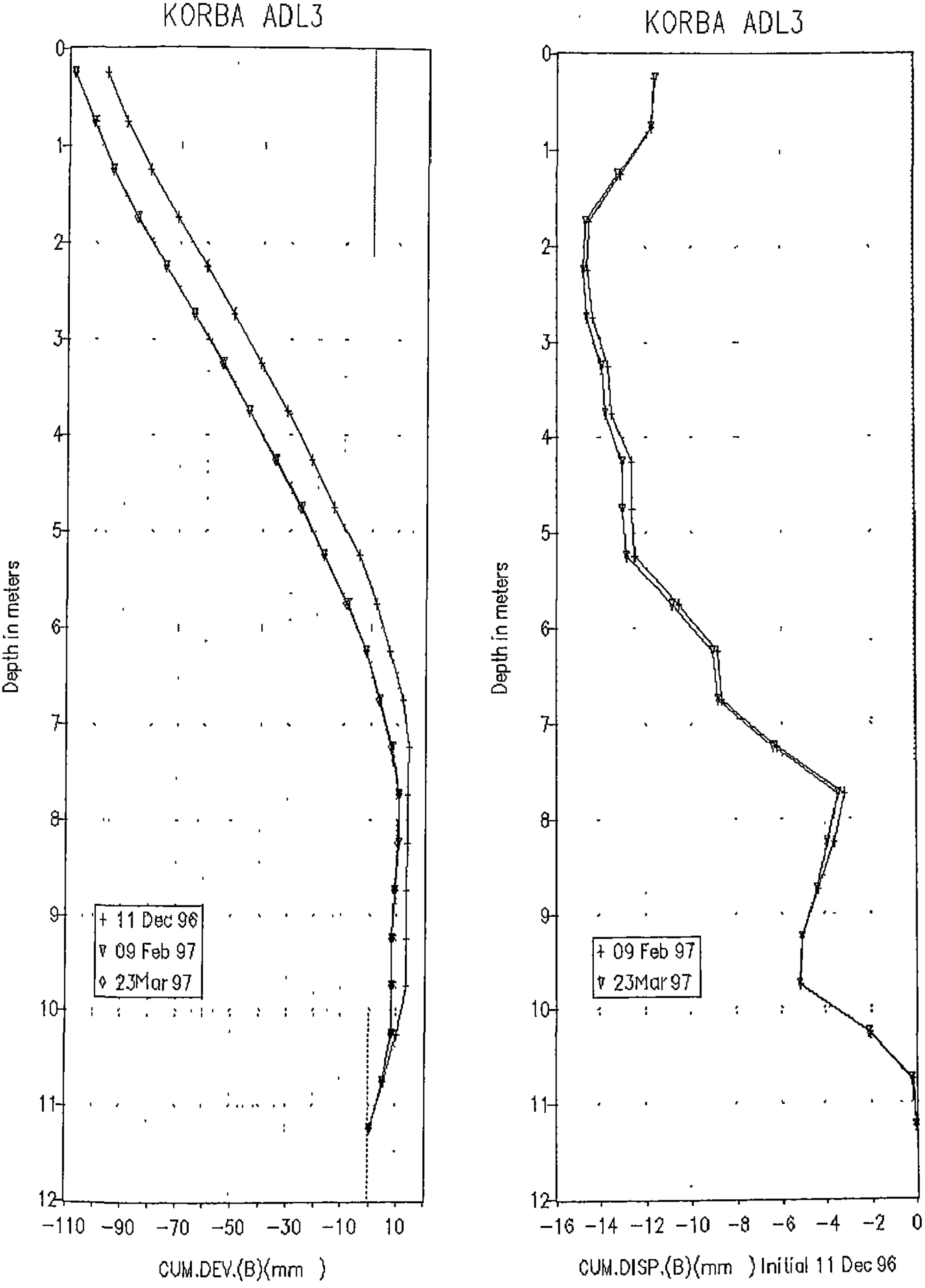

KORBA ADL3
Depth in meters
0
1
2
3
4
5
6
7
8
9
10
11
12
-110
-90
-70
-50
-30
-10
10
CUM.DEV.(B)(mm)
+ 11 Dec 96
▿ 09 Feb 97
◊ 23Mar 97
KORBA ADL3
Depth in meters
-16
-14
-12
-10
-8
-6
-4
-2
0
CUM.DISP.(B)(mm) Initial 11 Dec 96
+ 09 Feb 97
▿ 23Mar 97

Fig. 4.8 (b)

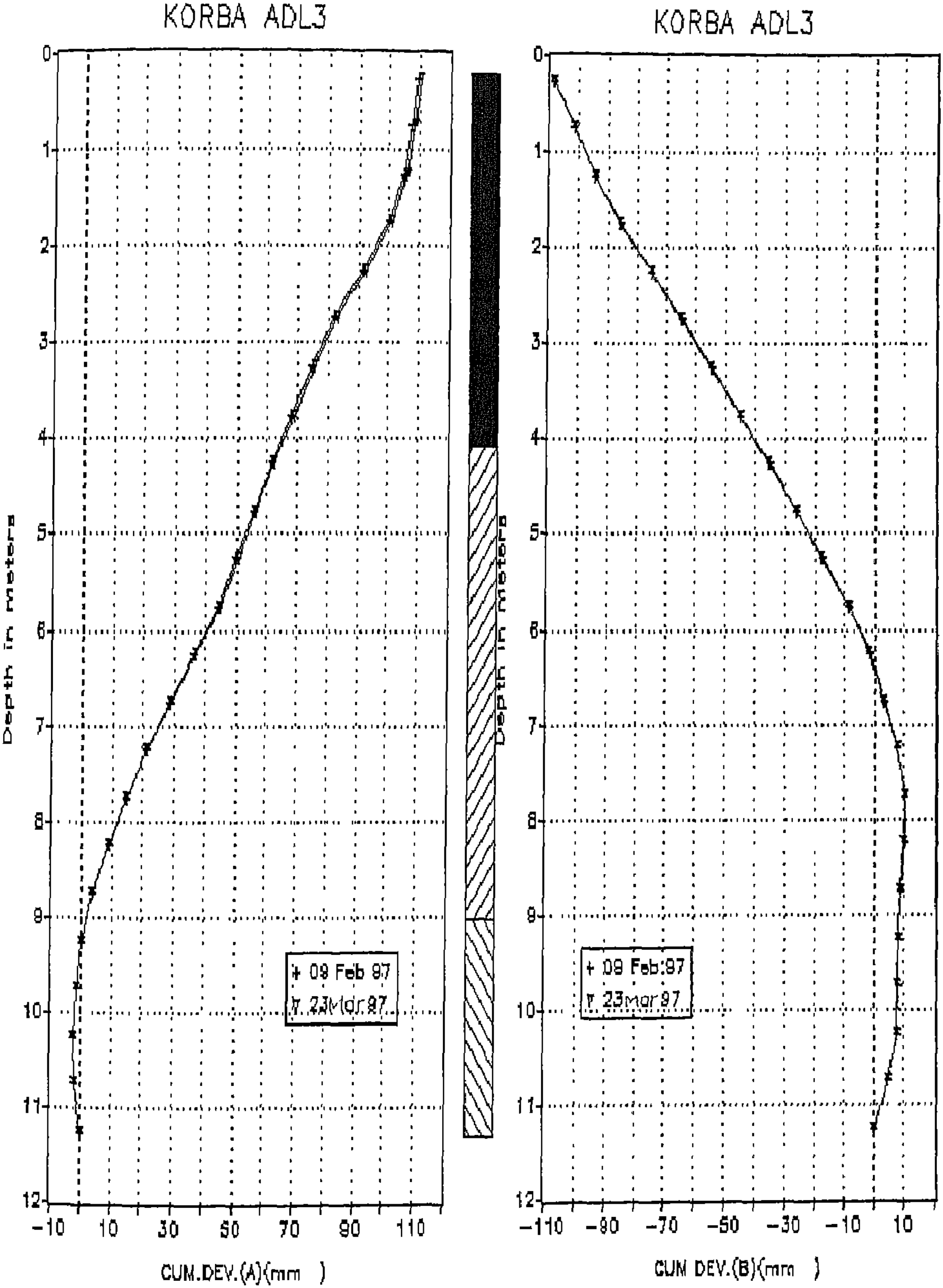

KORBA ADL3
Depth in meters
0
1
2
3
4
5
6
7
8
9
10
11
12
-10
10
30
50
70
90
110
CUM.DEV.(A)(mm)
+ 09 Feb 97
▿ 23Mar 97
KORBA ADL3
Depth in meters
-110
-90
-70
-50
-30
-10
10
CUM DEV.(B)(mm)
+ 09 Feb 97
▿ 23mar 97

Fig. 4.9(a)

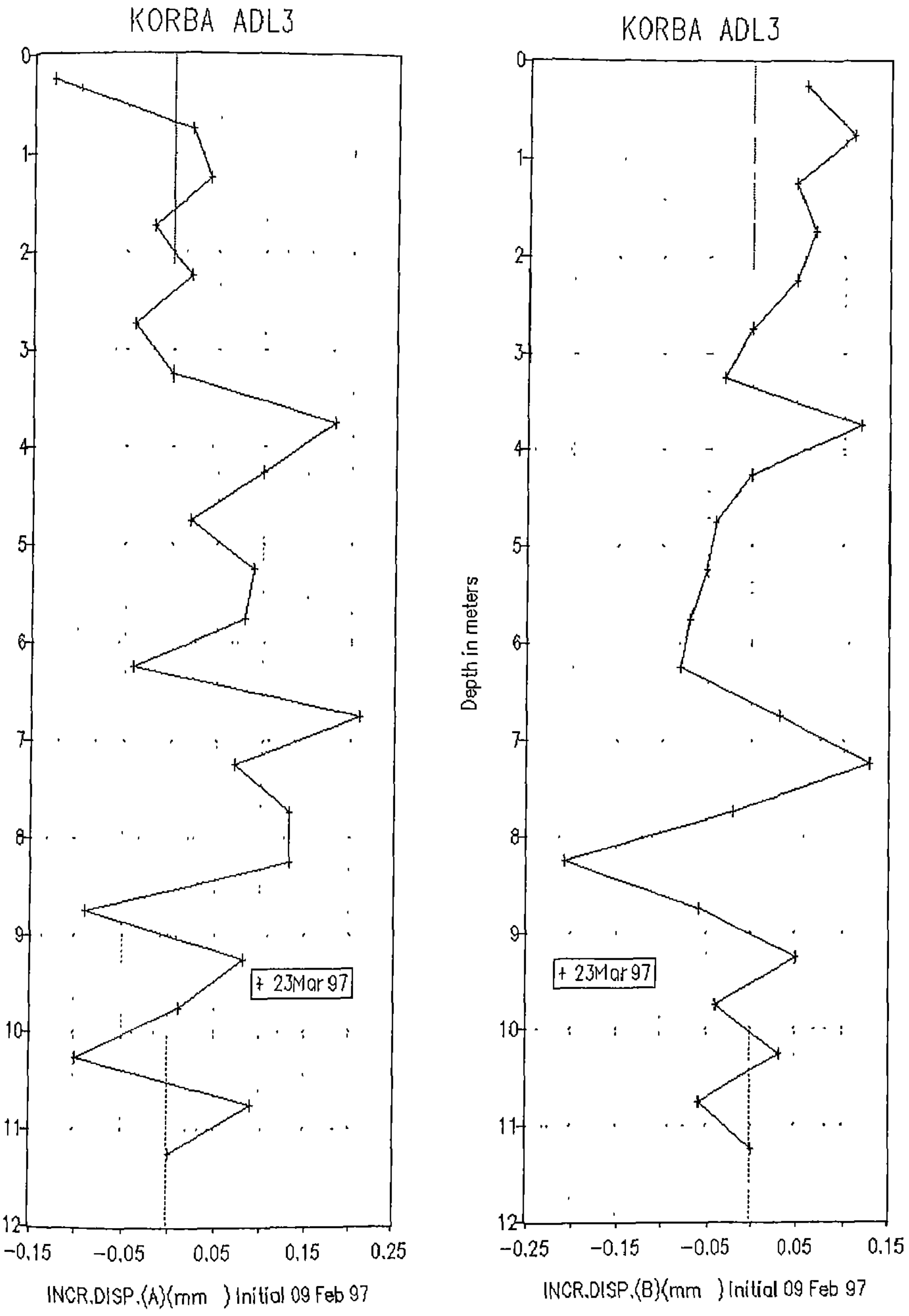

KORBA ADL3
0
1
2
3
4
5
6
7
8
9
10
11
12
-0.15
-0.05
0.05
0.15
0.25
INCR.DISP.(A)(mm) Initial 09 Feb 97
+ 23Mar 97
KORBA ADL3
Depth in meters
-0.25
-0.15
-0.05
0.05
0.15
INCR.DISP.(B)(mm) Initial 09 Feb 97
+ 23Mar 97

Fig 4.9(b)

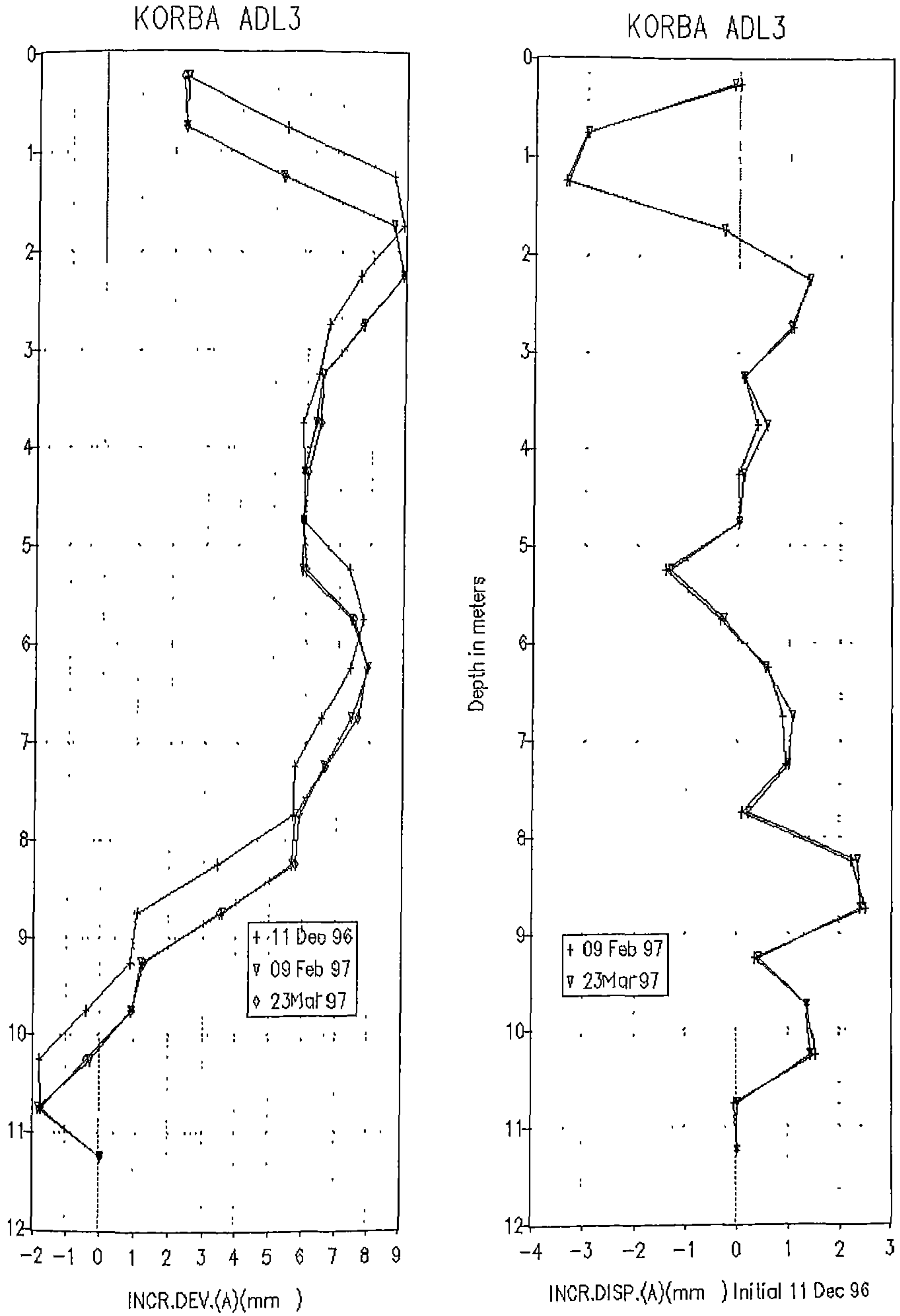

KORBA ADL3
0
1
2
3
4
5
6
7
8
9
10
11
12
-2
-1
0
1
2
3
4
5
6
7
8
9
+ 11 Dec 96
▽ 09 Feb 97
◇ 23Mar 97
INCR.DEV.(A)(mm)
KORBA ADL3
Depth in meters
-4
-3
-2
-1
0
1
2
3
+ 09 Feb 97
▽ 23Mar97
INCR.DISP.(A)(mm) Initial 11 Dec 96

Fig 4.10(a)

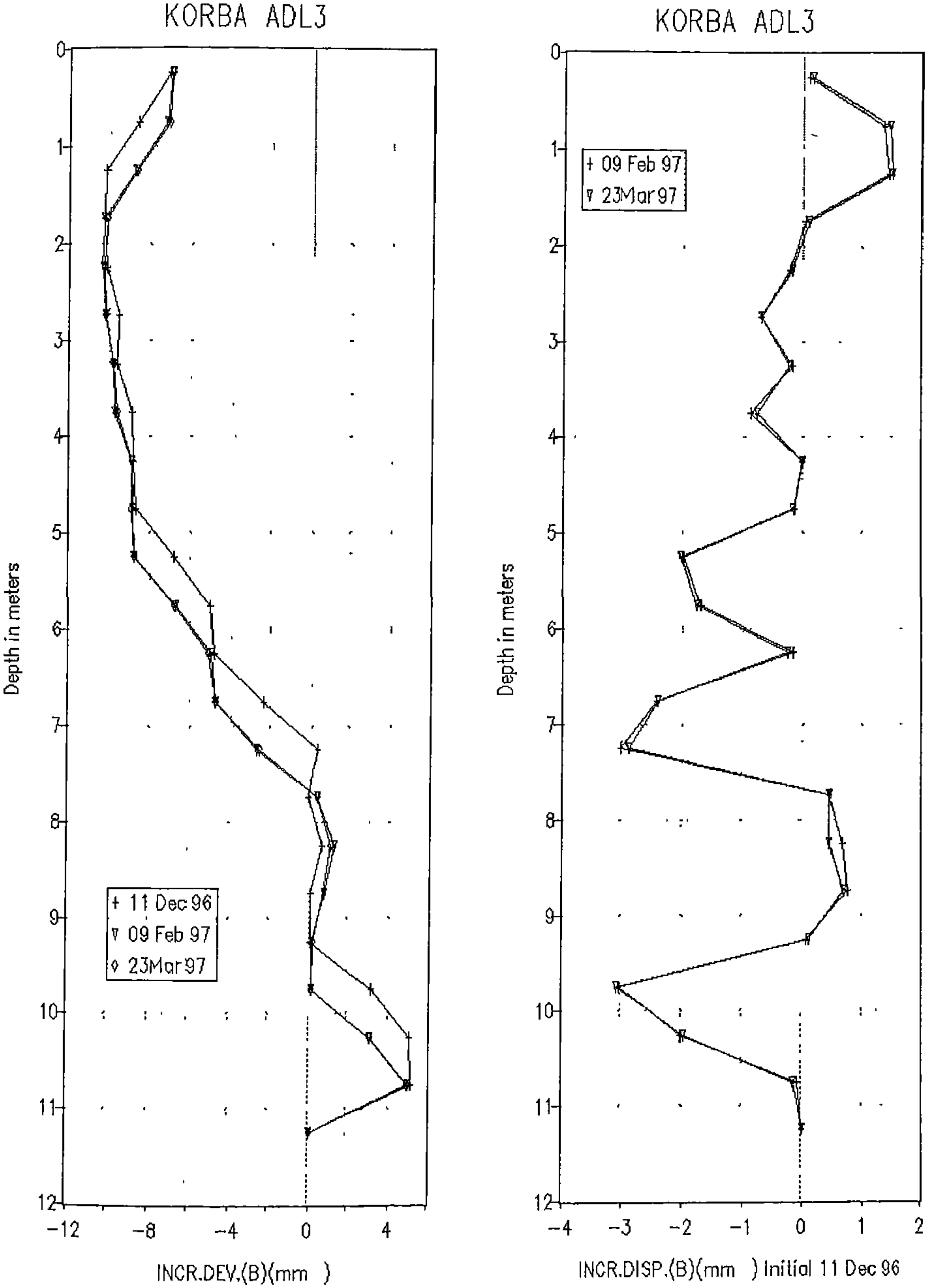

KORBA ADL3
Depth in meters
0
1
2
3
4
5
6
7
8
9
10
11
12
−12
−8
−4
0
4
INCR.DEV.(B)(mm)
+ 11 Dec 96
▽ 09 Feb 97
◇ 23Mar 97
KORBA ADL3
Depth in meters
−4
−3
−2
−1
0
1
2
INCR.DISP.(B)(mm) Initial 11 Dec 96
+ 09 Feb 97
▽ 23Mar 97

Fig. 4.10(b)

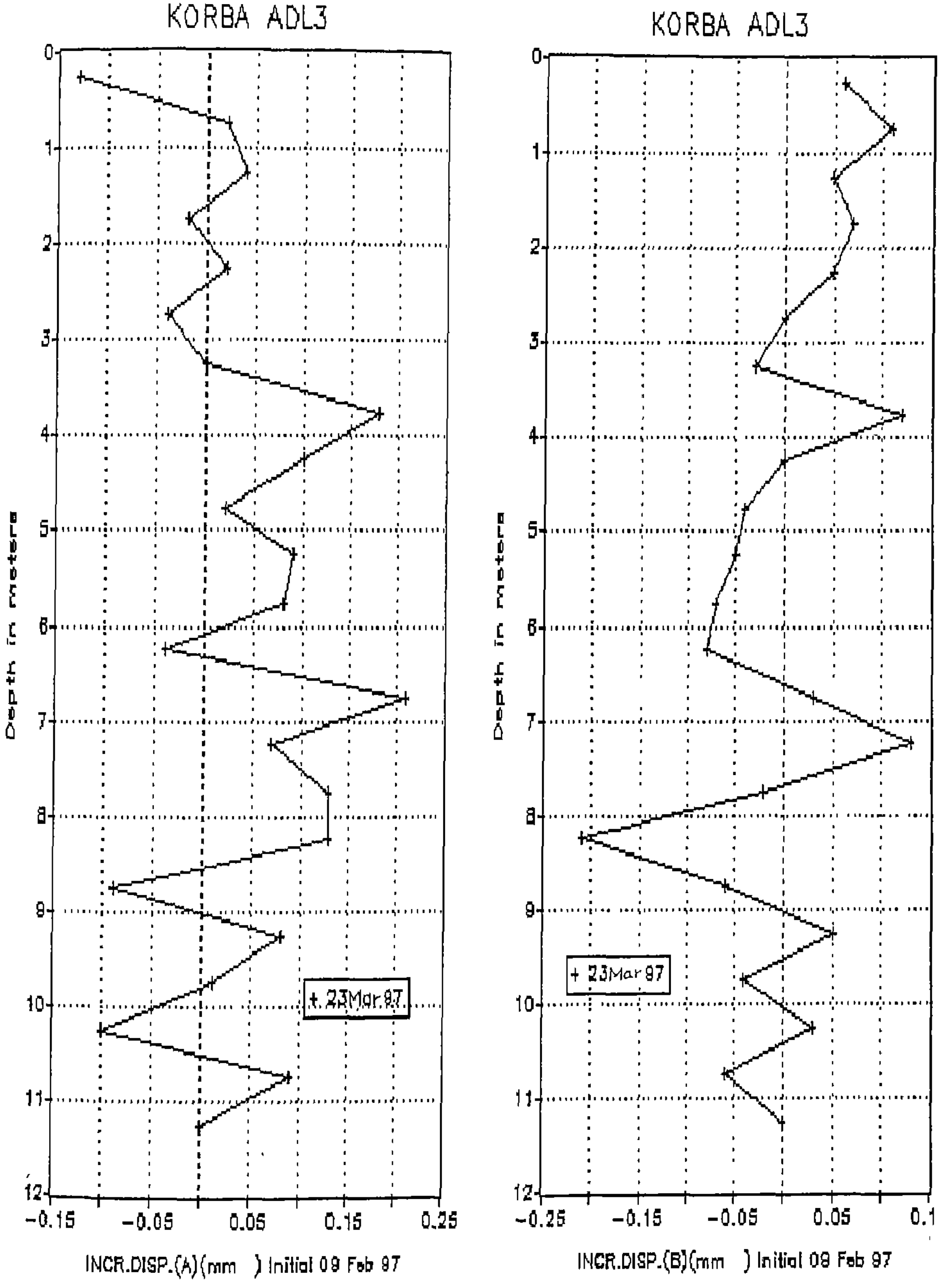

KORBA ADL3
Depth in meters
0
1
2
3
4
5
6
7
8
9
10
11
12
-0.15
-0.05
0.05
0.15
0.25
INCR.DISP.(A)(mm) Initial 09 Feb 97
+ 23Mar 97
KORBA ADL3
Depth in meters
-0.25
-0.15
-0.05
0.05
0.1
INCR.DISP.(B)(mm) Initial 09 Feb 97
+ 23Mar 97

Fig 4.11

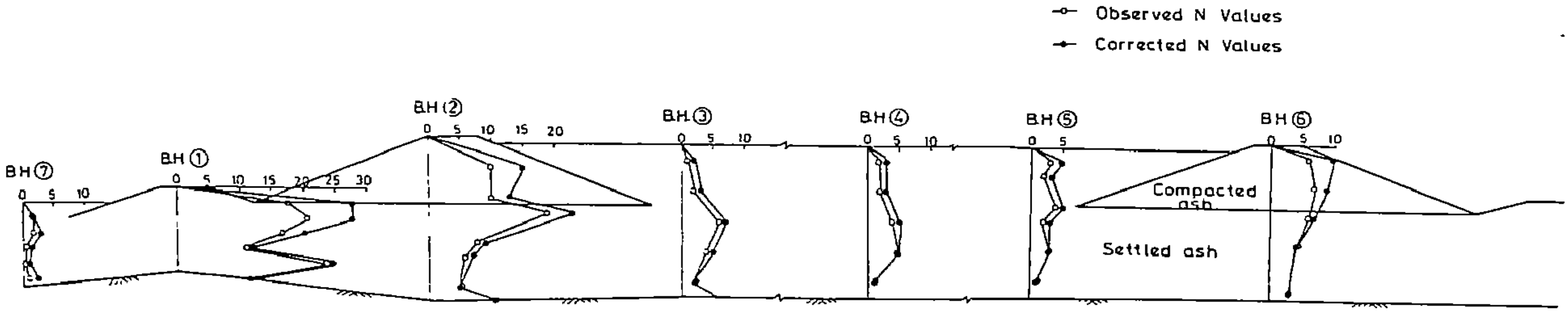
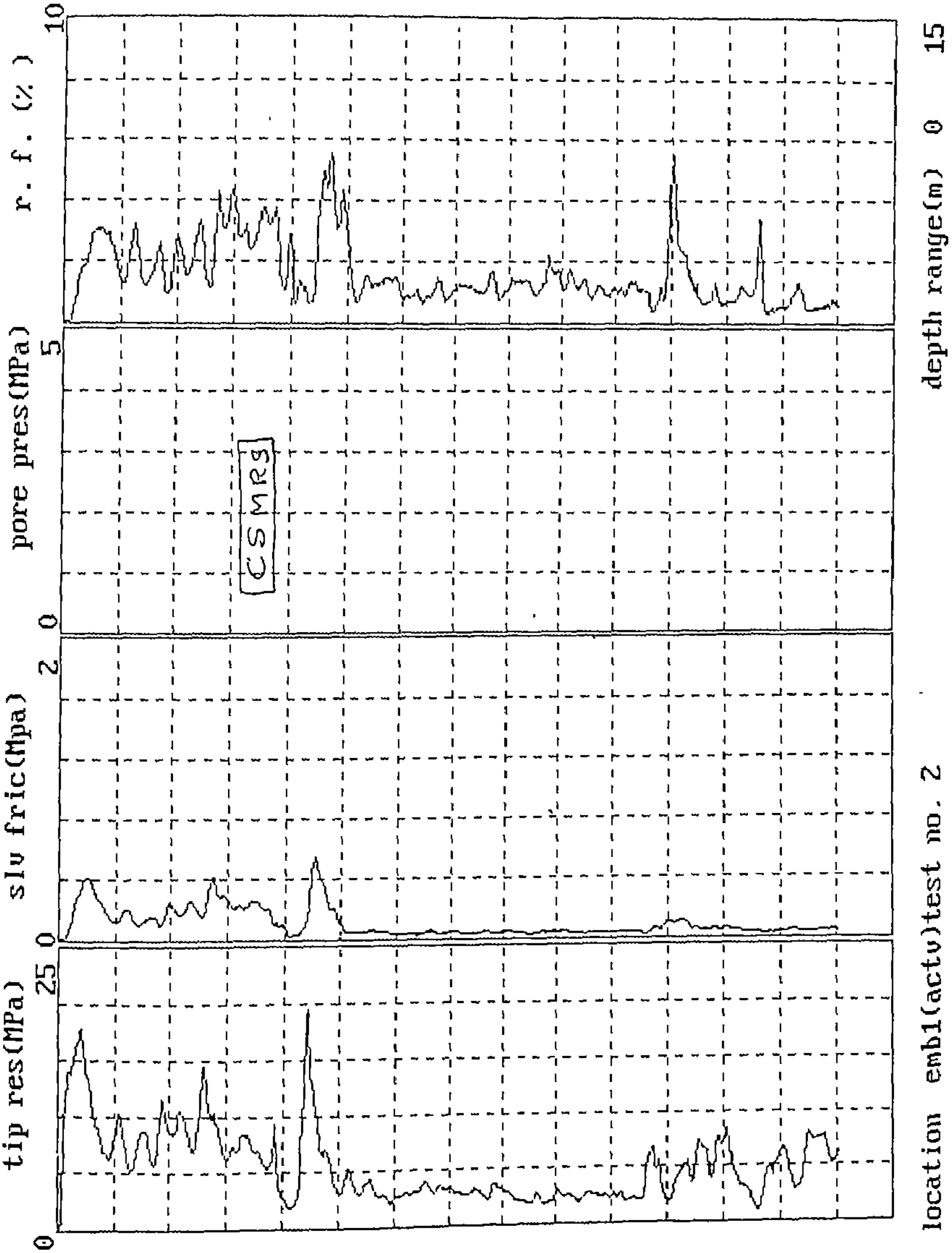
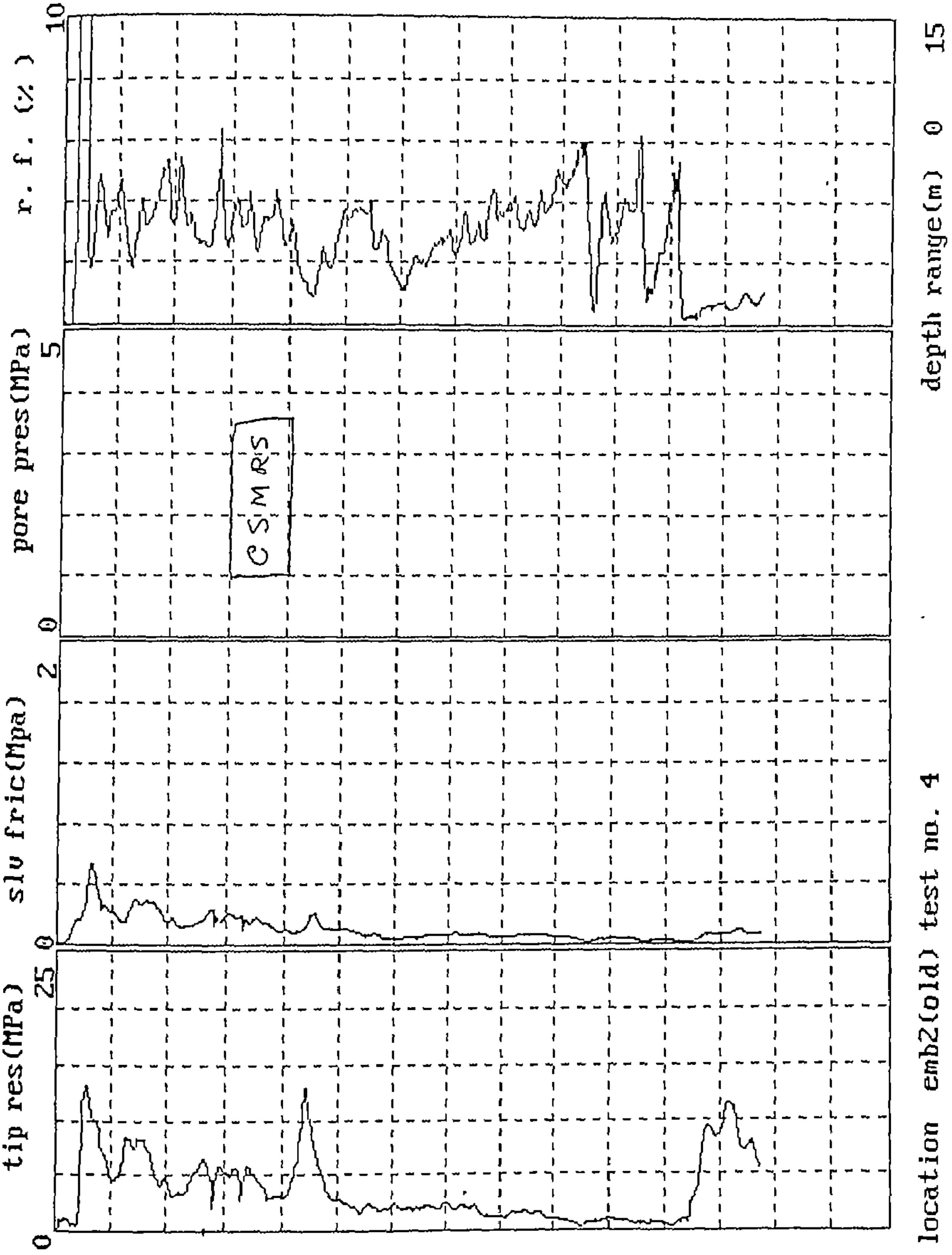
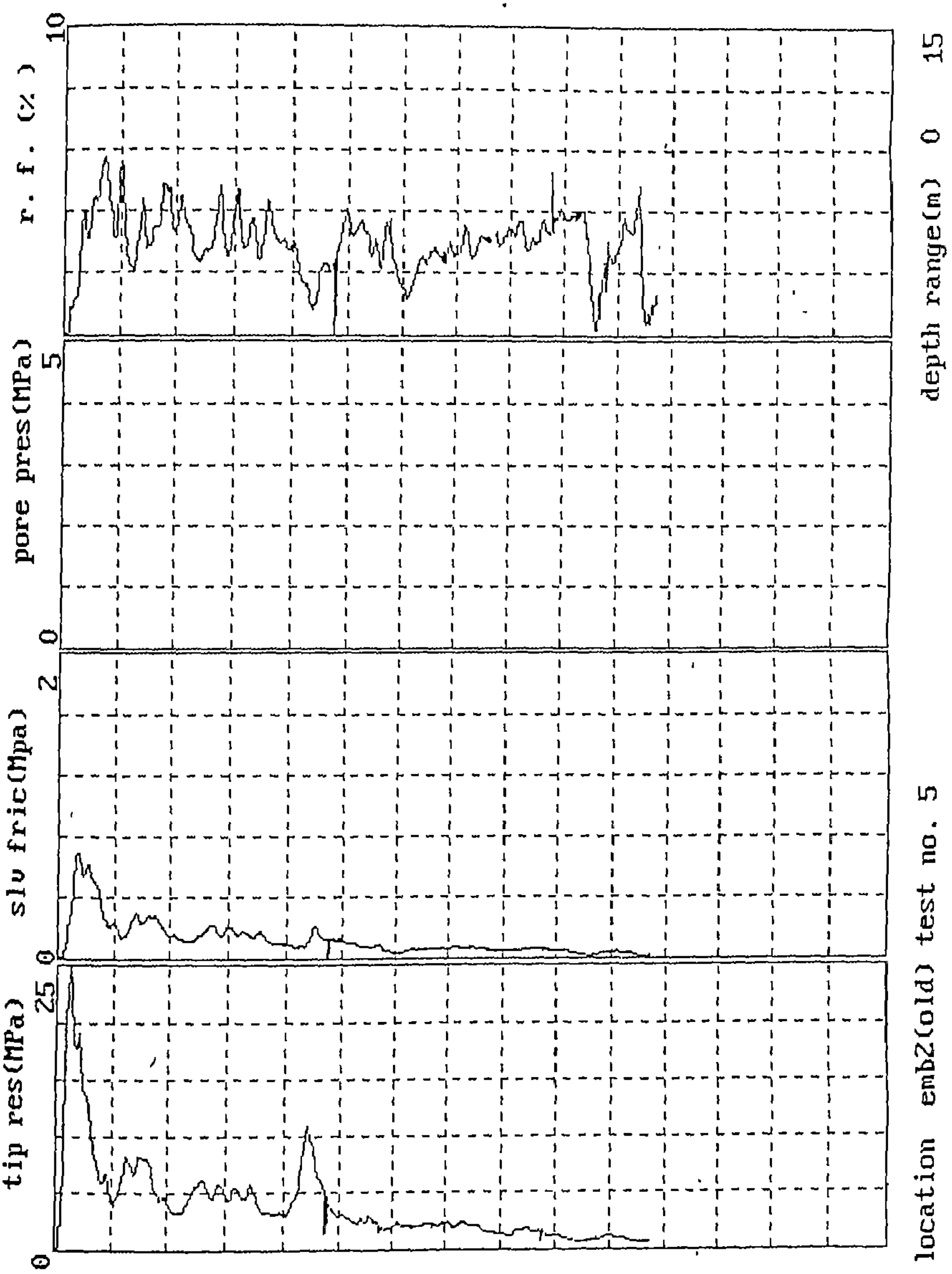


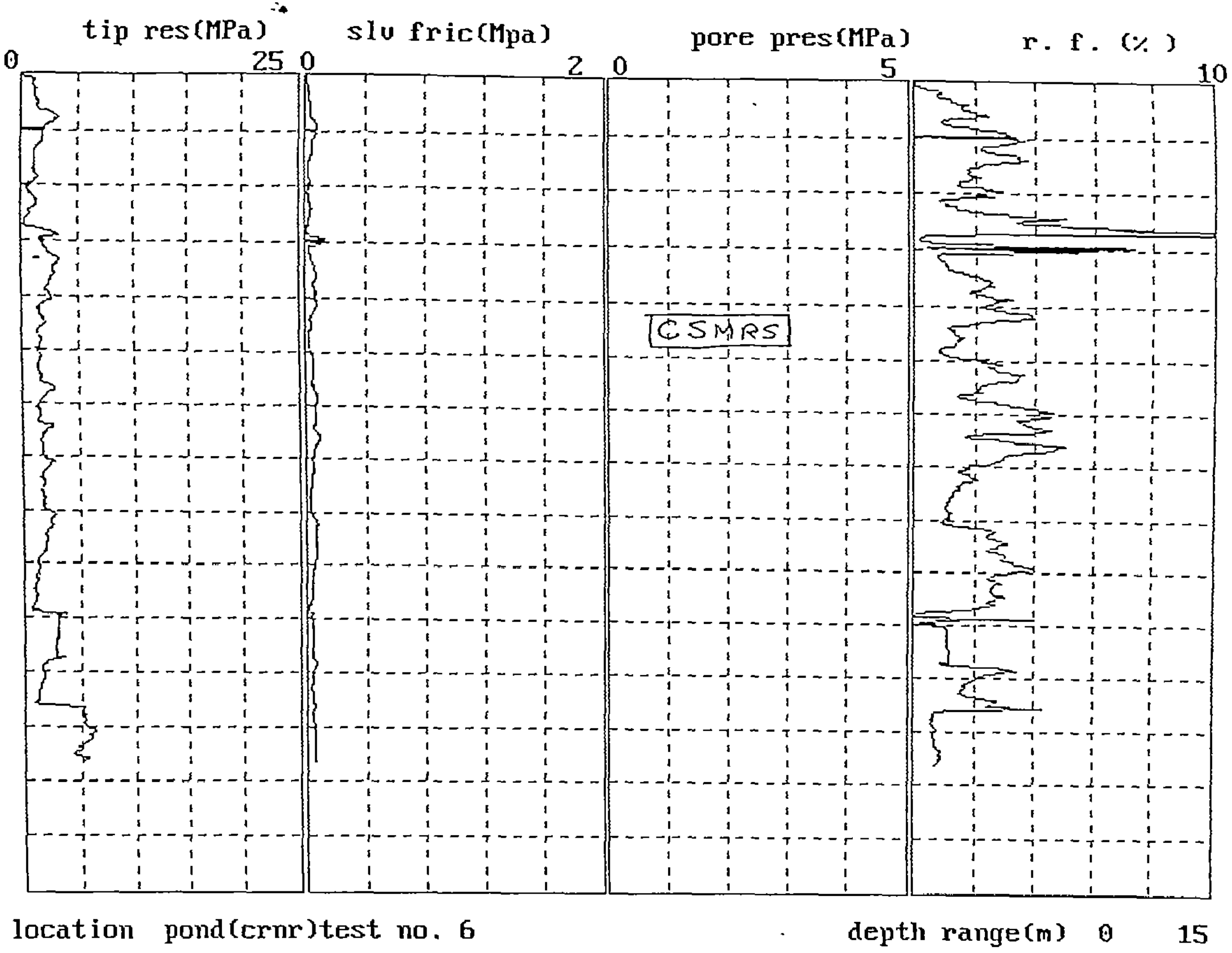
FIG 4.12 STANDARD PENETRATION TEST AT BTPS-ASH DYKE

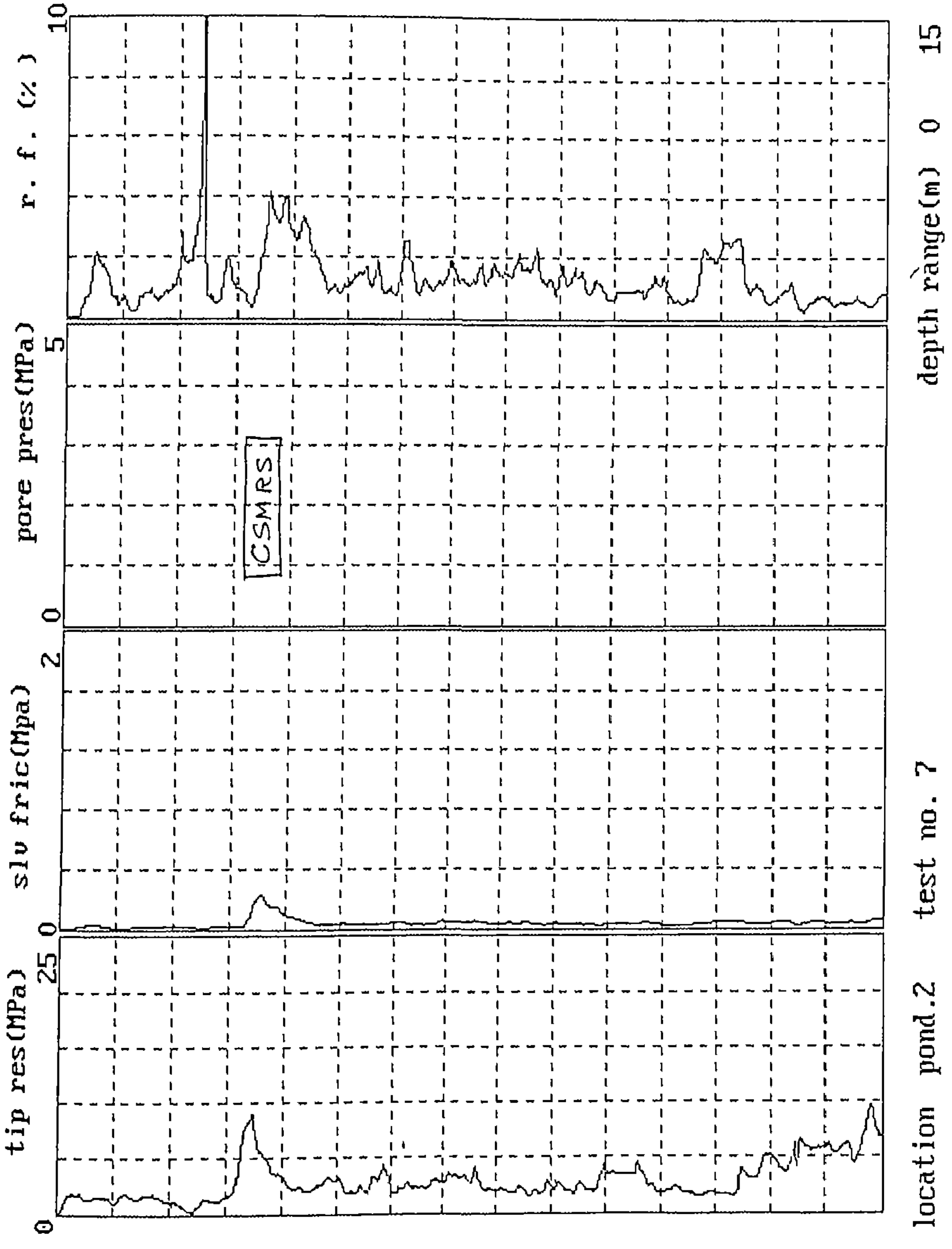

tip res(MPa)
0
25
slv fric(Mpa)
0
2
pore pres(MPa)
0
5
CSMRS
r. f. (%)
10
location emb1(actu)test no. 2
depth range(m) 0 15

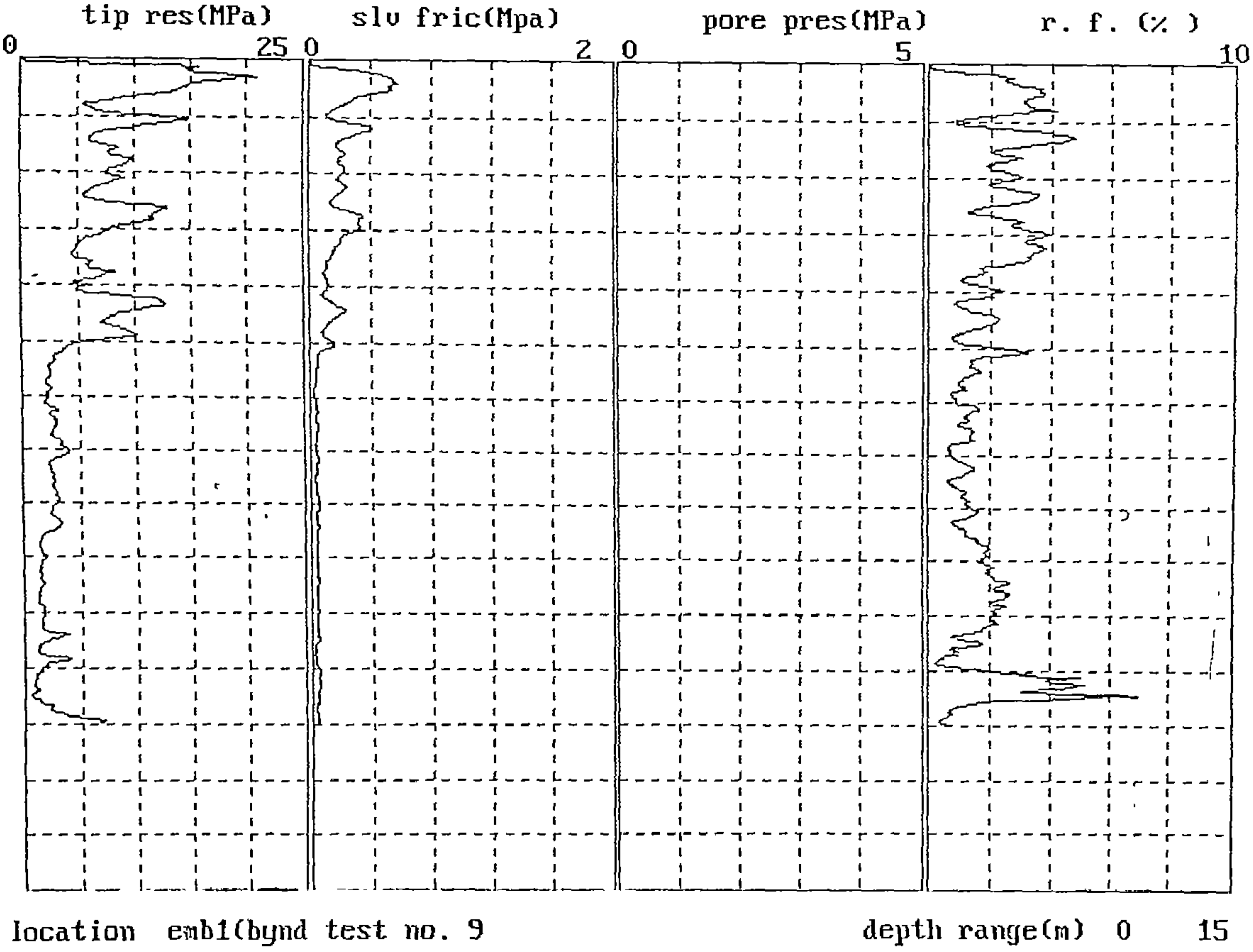

r. f. (%)
10
pore pres(MPa)
5
0
CSMRS
slv fric(Mpa)
2
0
tip res(MPa)
25
0
depth range(m) 0 15
location emb2(old) test no. 4


tip res(MPa)
0
25
slv fric(Mpa)
0
2
pore pres(MPa)
0
5
r. f. (%)
10
location emb2(old) test no. 5
depth range(m) 0 15

(c)


tip res(MPa)
0
25
slu fric(Mpa)
0
2
pore pres(MPa)
0
5
r. f. (%)
10
CSMRS
location pond(crnr)test no. 6
depth range(m) 0 15


tip res(MPa)
0
25
slv fric(Mpa)
0
2
pore pres(MPa)
0
5
CSMRS
r. f. (%)
10
location pond.2
test no. 7
depth range(m) 0 15


tip res(MPa)
0
25
slu fric(Mpa)
0
2
pore pres(MPa)
0
5
r. f. (%)
10
location emb1(bynd test no. 9
depth range(m) 0 15

(g)

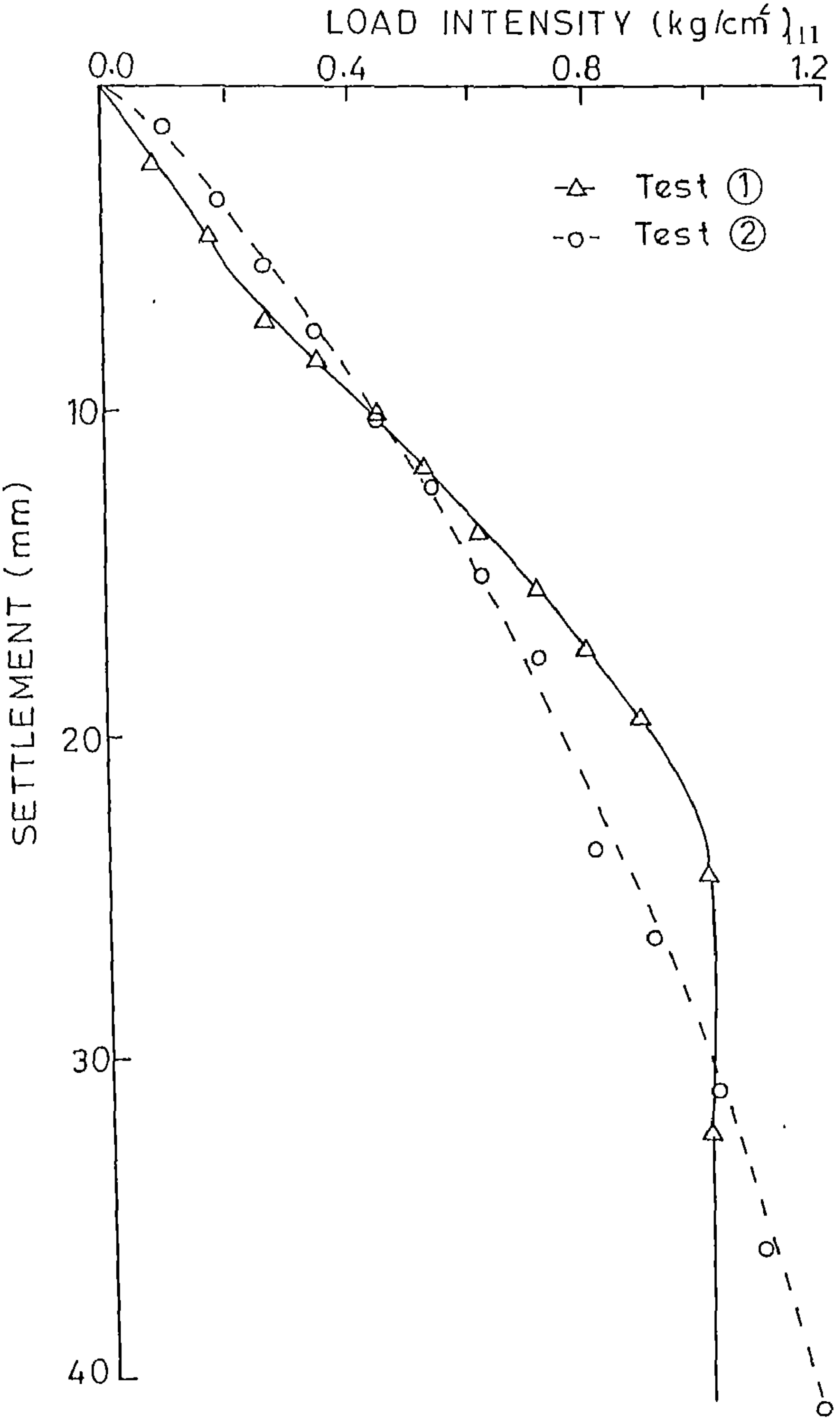


FIG.4.14 PLATE LOAD TEST AT BTPS

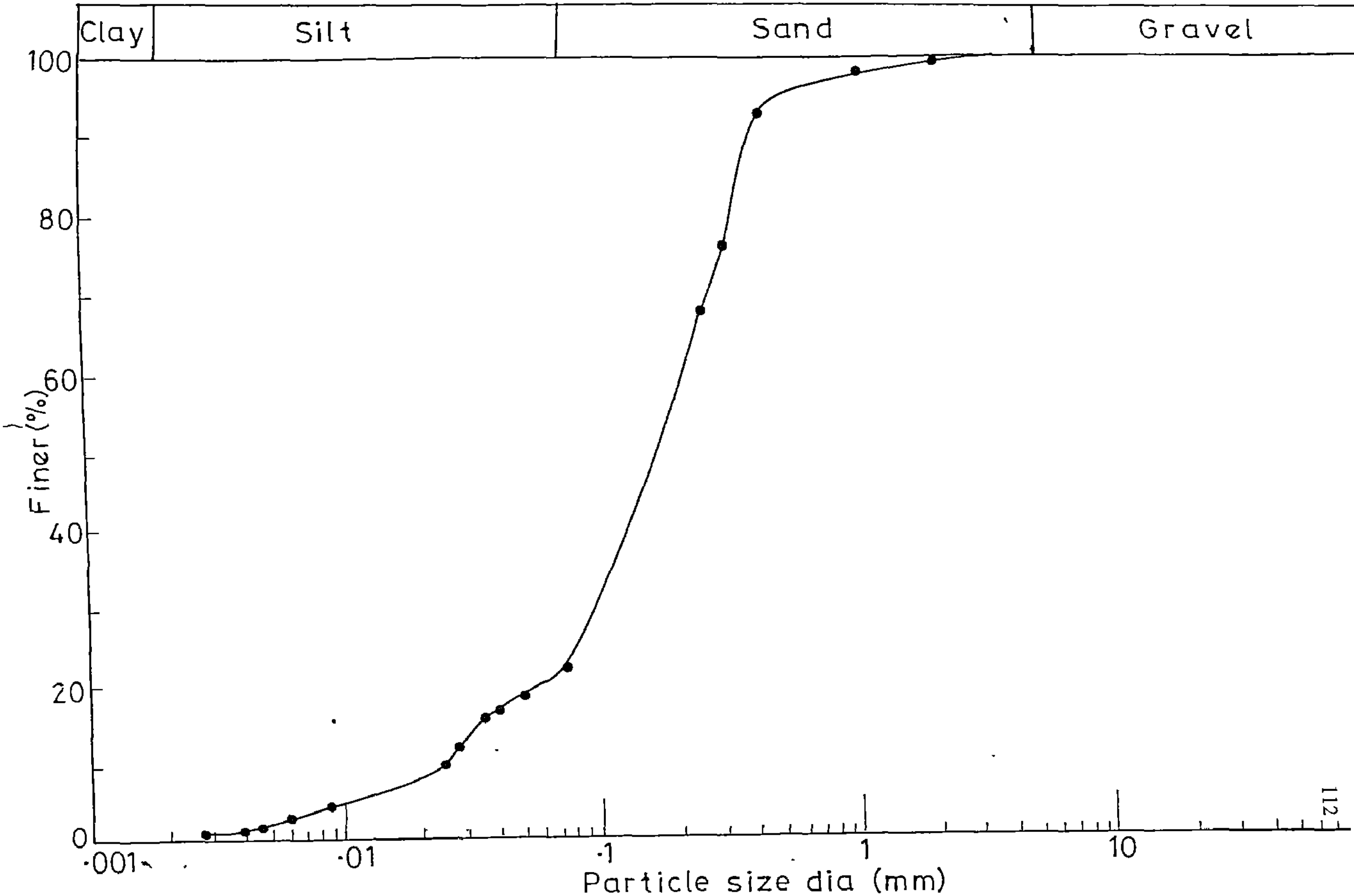


FIG.4.15 GRAIN SIZE DISTRIBUTION OF BTPS-ASH

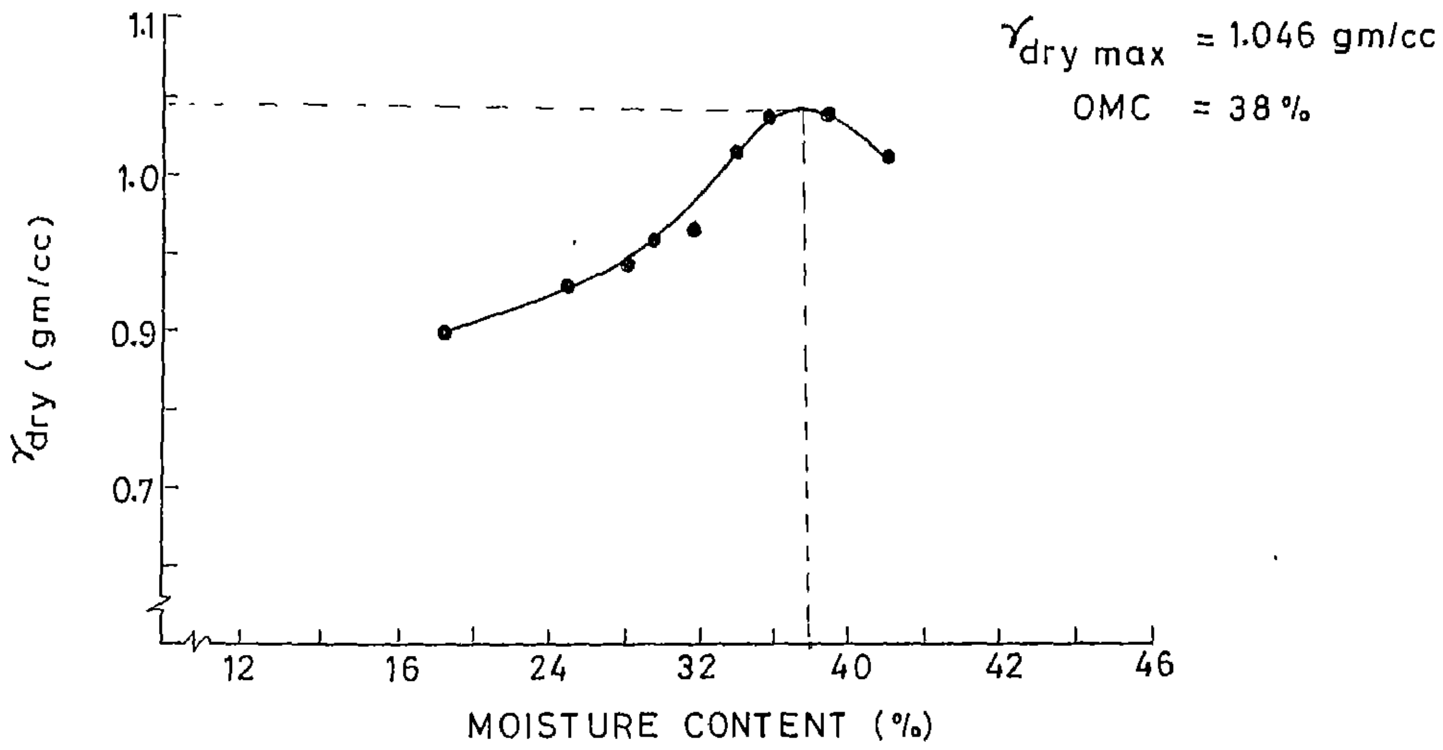


FIG.4.16 STANDARD PROCTOR TEST FOR BTPS-ASH

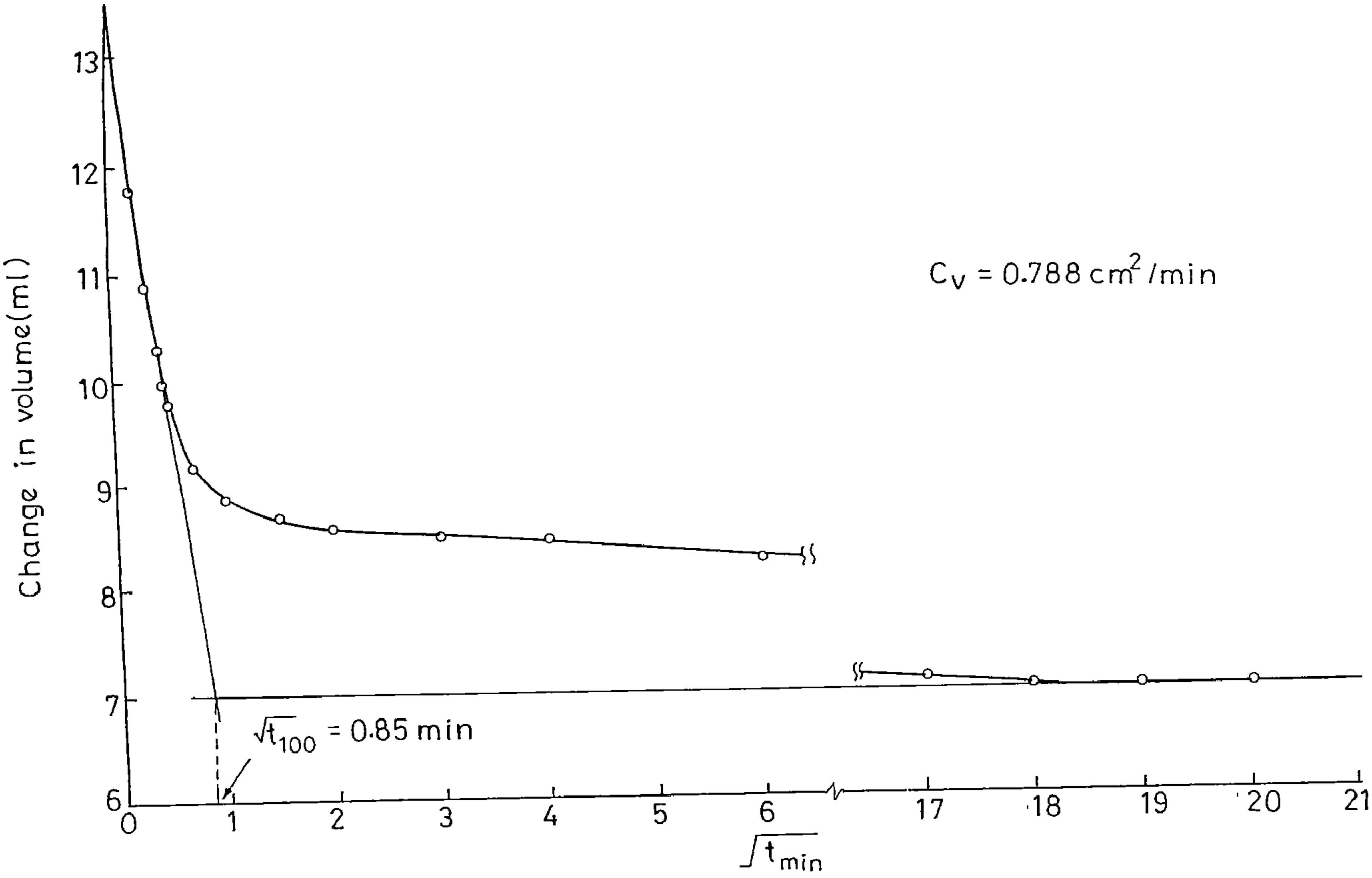


FIG.4.17 C_v-DETERMINATION OF BTPS-ASH SAMPLE (95 %)

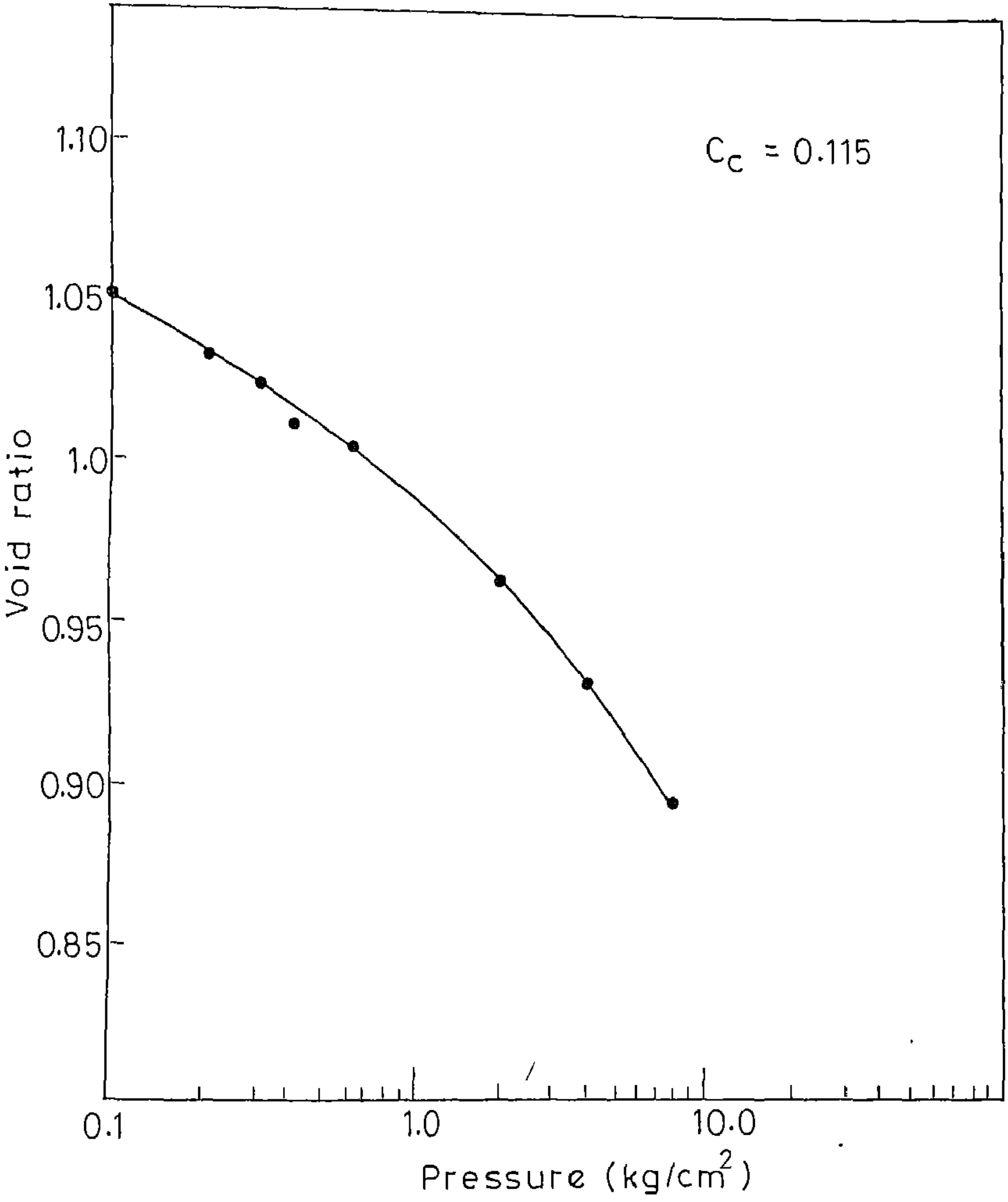


FIG.4.18 e-log P CURVE FOR BTPS ASH SAMPLE (95 %)

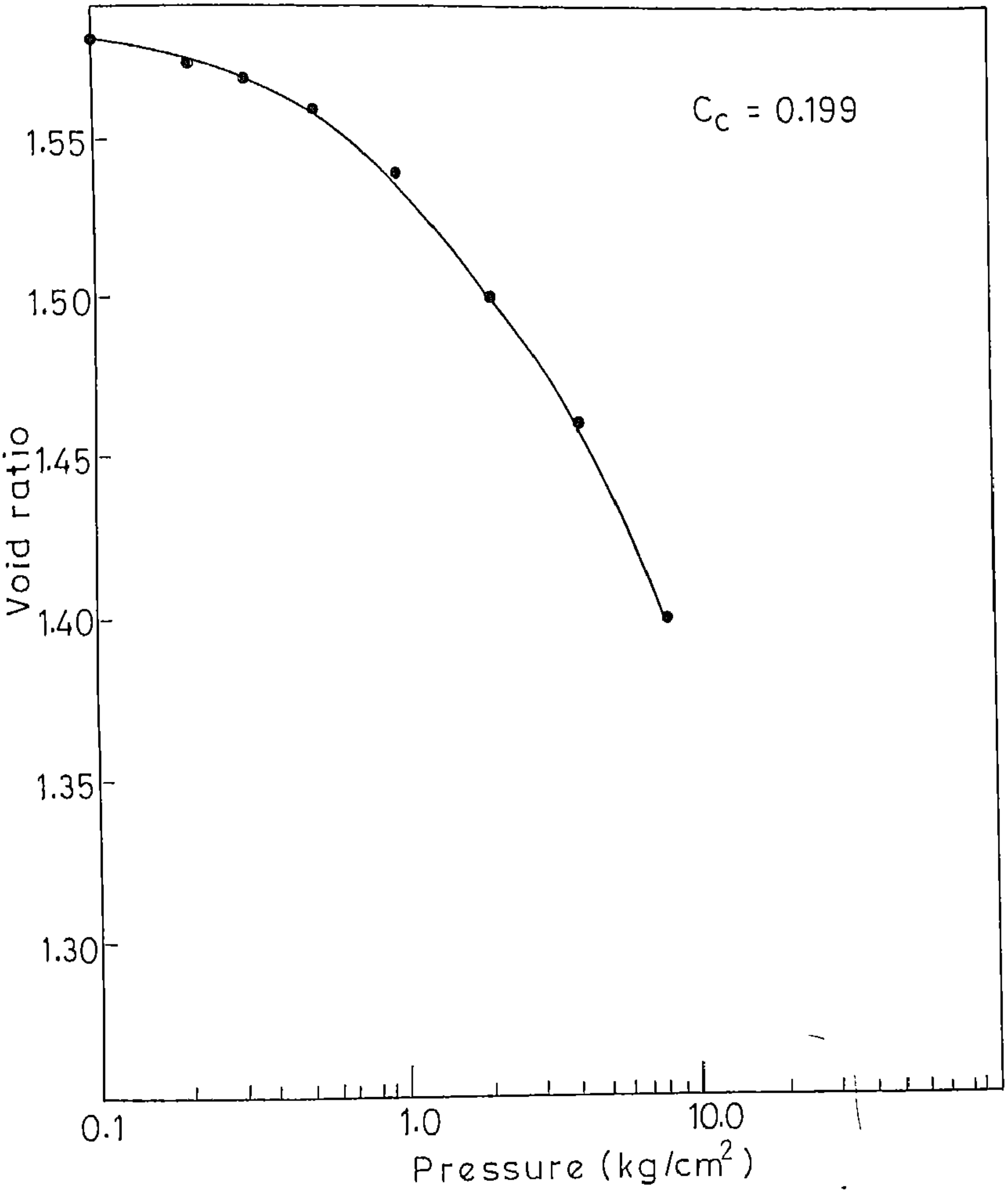


FIG.4.19 e-log P CURVE FOR BTPS-ASH SLURRY

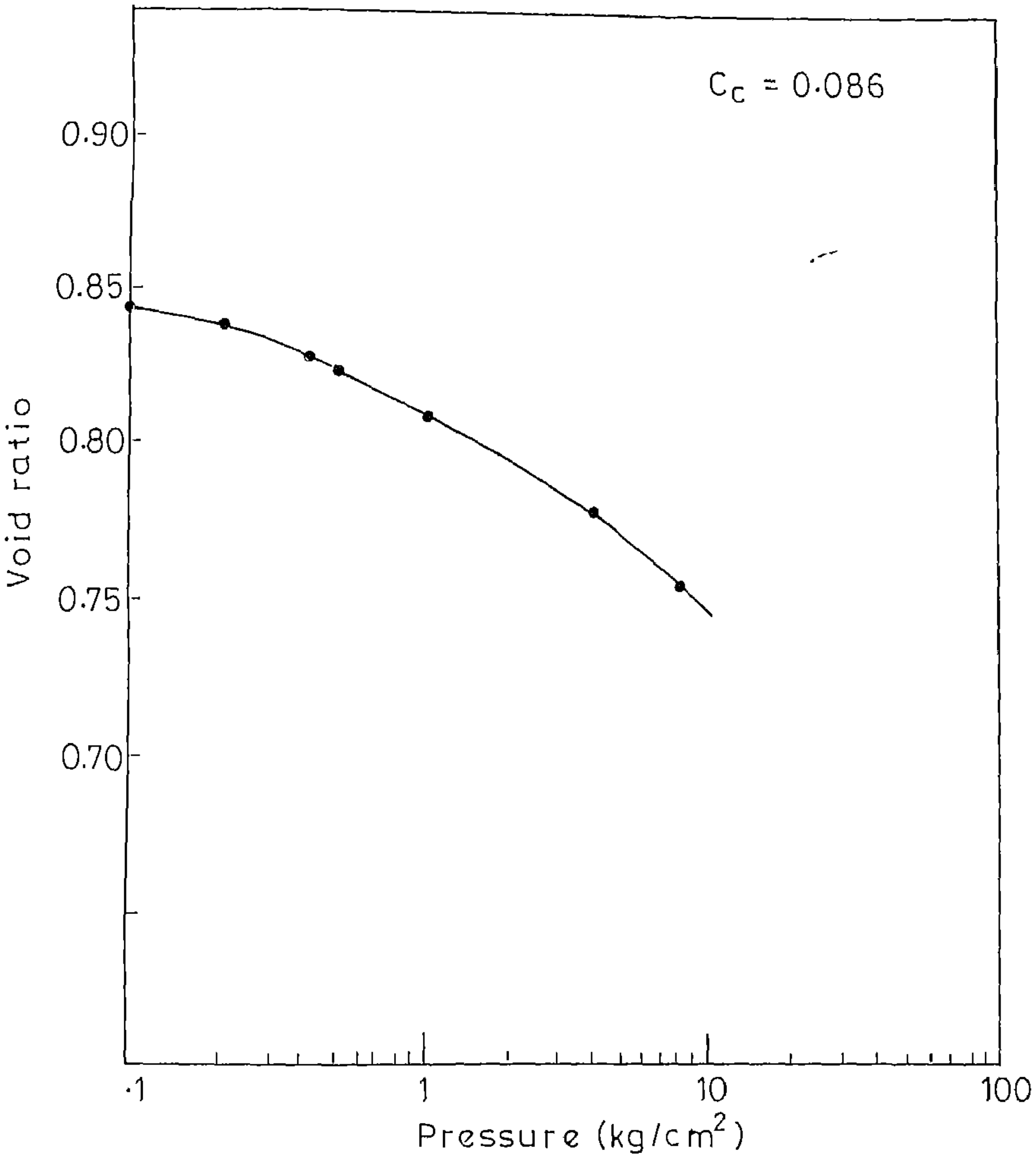


FIG.4.20 e-log P CURVE FOR KSTPP-ASH SLURRY

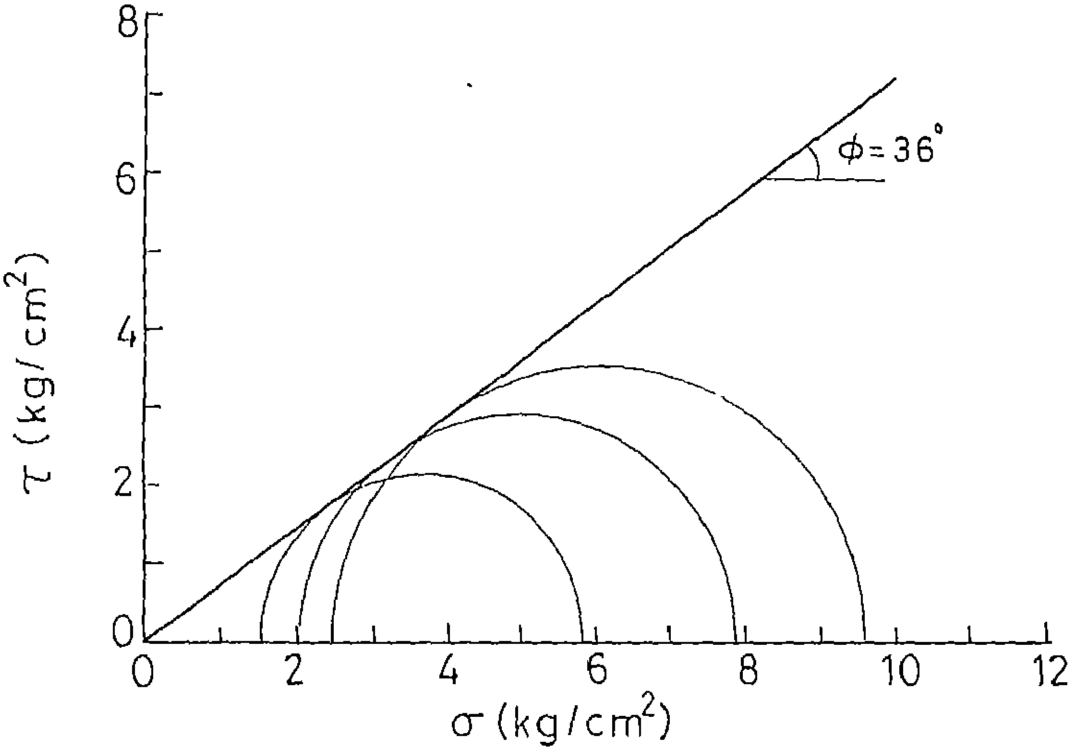
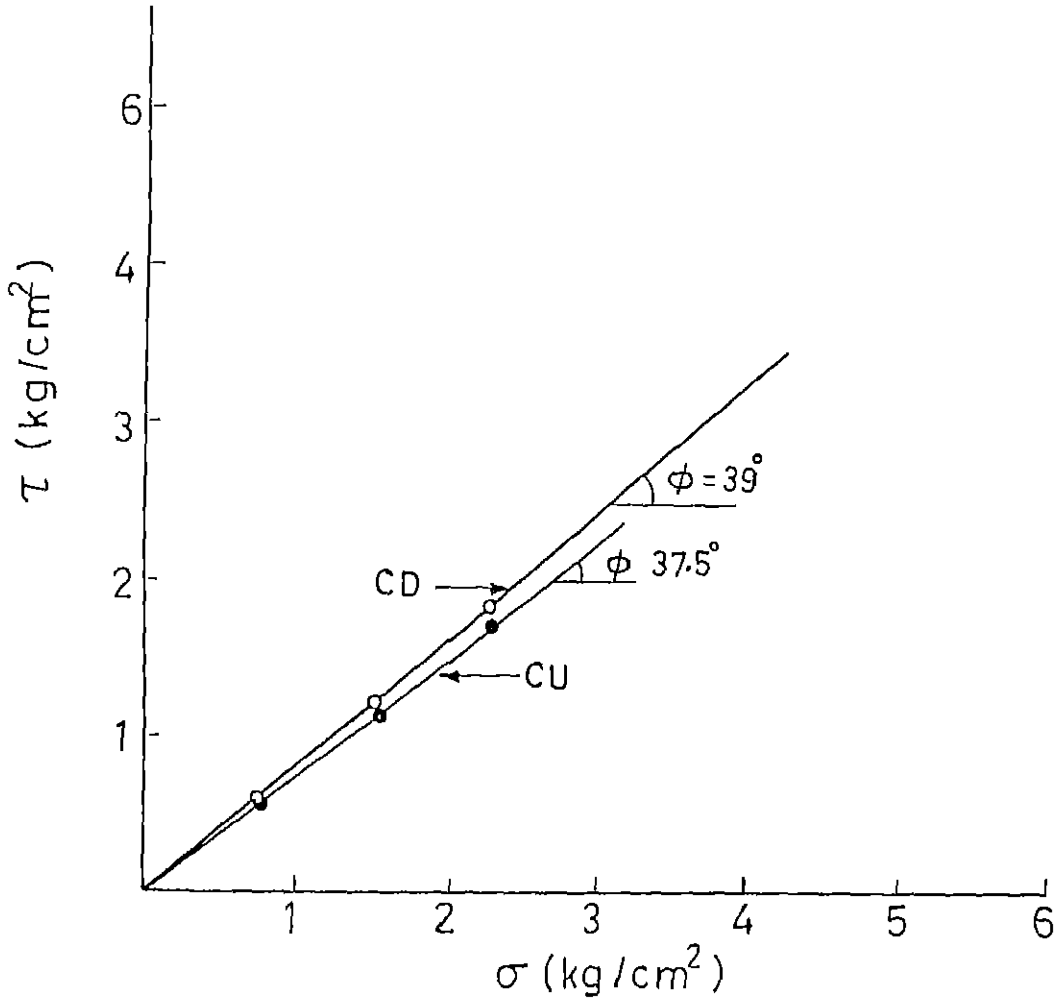


FIG.4.21 TRIAXIAL TEST ON BTPS ASH SAMPLE (95%)



FIG·4.22 DIRECT SHEAR TEST ON BTPS-ASH (95 %)

Chapter 5

Results and Discussion

5.1 General

The correlation between tip resistance (q_c) and N values for pond ash have been established. Record of plate load test conducted by NTPC in the ash pond of BTPS has also been analyzed in this chapter for comparative study Settlements and Lateral movement of ash dyke embankment have been monitored, during and after ash filling operation, in a lagoon of KSTPP ash dyke. Piezometer readings taken at several stages of embankments have been analyzed to check its functionality during ash disposal operation. Finally the improvement in shear strength parameters of pond ash have been investigated through various field and laboratory tests, and it's effect on the stability of the ash dykes have also been studied in this chapter.

5.2 Study of Shear Strength Parameter

Various field and laboratory tests, as discussed in section 4 4, were conducted to evaluate shear strength parameter of ash at the two selected sites of KSTPP and BTPS ash dyke. Standard penetration tests were conducted at KSTPP ash dyke while piezocone and SPT both were conducted at BTPS ash dyke The summary of angle of internal friciton, as obtained from various filed and laboratory mehtods are given in Table 5.1

5.2.1 KSTPP Ash Dyke

Angle of internal friction was evaluated using standard penetration test A marked improvement in the ϕ value of settled ash have been observed, since initial stages of construction of ash dyke From the Table 5 5, the ϕ of settled ash was initially 28° and after six years of dyke construction, it has improved to a minimum of 31° for first raising while at it has further improved to 33° in the second raising. The marked improvement in the strength of the settled ash beneath the ash embankment is partly due to cosolidation of the settled ash and partly due to imposition of the load of compacted ash fill. Construction equipment operation over the settled ash during the embankment construction may have also resulted in some compaction of the material. The improvement have further enhanced the chances of further raising of ash dyke by upstream method of construction.

5.2.2 BTPS Ash Dyke

At BTPS ash dyke both piezocone and SPTs were conducted to evaluate the change in the settled ash properties, especially beneath the embankment raising. The test details and analysis of angle of internal friction obtained by piezocone for this site has been given in section 4 5 2 . Dayal et al (1982) have reported that the Trofimenkov method provides an over conservative estimate of ϕ while both Schmertmann and Meyerhof methods provide a reasonable agreement with directly measured values in the field for various soils. Therefore the ϕ values reported in the present discussion for piezocone is limited to the lowest of the two values estimated from Schmertmann and Meyerhof methods With piezocone, a ϕ values (refer table 4 7) for the compacted ash embankment were found to be 35° at southern end and 36° at northern end while with SPT (refer table 4 6) for the same locations was found to be 30° and 33° respectively.

For settled ash, the ϕ value obtained from piezocone exhibited a minimum value of 31 5° by Meyerhof method and a minimum of 28° by SPT From the Table 4 6 and 4.7 , it is clear that ϕ values on the slurry discharge end (northern end) is comparatively higher than on the water outlet end (southern end) This may be attributed to the decreasing grain size of the settled ash from discharge end to the water outlet end. The friction ratio also justifies the above finding as the friction ratio near the slurry discharge end is 1.0 which falls in the range of clean sand with no plastic fines while it is 3 near the water outlet end depicting the range of clayey sand

The ϕ values obtained from plate load test at different locations of pond exhibits a value of 32° and 30° for settled ash (refer table 4 8) The value may be higher owing to the limitations of plate load test

The laboratory test results are also in conformance with the field test results for compacted ash. Since embankments are constructed at 95 % of Standard Proctor Density and since ash is cohesionless material, therefore consolidated drained tests were performed on samples prepared at 95 % of Standard Proctor Density in triaxial test apparatus as well as in direct shear test apparatus The ϕ values obtained are 36° from direct shear test and 39° from triaxial test Applying a correction factor of 0.9 to the direct shear result a ϕ value of about 36° is found for both of these tests.

The cone resistance (q_c) obtained from piezocone and the N-values has been compared for those test locations which were close enough to represent the same type of ash.

Table 5.1 Summary of the angle of internal friction obtained through various field and laboratory test for BTPS ash

zone	Meyerhof max-min	Schmertmann max -min	SPT max-min	Plate load test	Direct Shear test		Triaxial test
					drained	undrained	
compacted ash	36°-35°	45°-39°	33°-30°	–	39°	37.5°	36°
settled ash	34°-31.5°	39°-31°	30°-28°	32°-30°	–	–	–

Table 5.2 q_c / N ratio for ash

Northern Embankment :

Depth	Material	q_c (kg/cm^2)	N-value	q_c/N
0-5.0	compacted ash	75	10	7 5
5 0-8 0	settled ash	30	7	4 3
8.0-10.5	settled ash	25	5	5 0
10.5-14.0	soil	40	11	3 6

Southern Embankment :

Depth	Material	q_c (kg/cm^2)	N-value	q_c/N
0-5.0	compacted ash	50	7	7 14
5 0-9.0	settled ash	20	4	5.0
9.0-11.5	settled ash	10	2	5 0
11.5-14.o	soil	75	33	2 3

Settled ash (at the end of ramp) .

Depth	Material	q_c (kg/cm^2)	N-value	q_c/N
0-3.0	compacted ash	15	2	7 5
3-12.5	settled ash	25	5	5 0
12.5-15	soil	50	19	2 6

Table 5.3 Correlation between qc and N [Schmertmann (1970)]

Soil Type	q_c / N *
Clean , fine to medium sands and sligtly silty sands	3.5
Coarse sands and sands with little gravel	5 0
Sandy gravel and gravel	6.0

* *Units of q_c are kg/cm^2* units of N are blows/ft

It is observed from the above table that q_c/N ratio for ash varies from 5 to 7. From Table 5 3, ash can be classified as coarse sands and sands with little gravel which is partially acceptable. Generally ash exhibits properties of silty sand but presence of unburnt carbon in the disposed ash may increase the q_c / N ratio

5.3 Settlement

Generally consolidation occurs at a very slow rate, sometimes over several months or years, but in case of embankment constructed of pond ash, consolidation occurs within few months. Consolidation can be a very important design factor because, in addition to causing damage to the drainage pipes and structures, it can affect several aspects of the upstream method of construction of ash disposal facility.

Settlement of the pond ash due to the construction of embankment over it, was observed with the help of cross arm settlement plates installed at KSTPP and BCPP ash dykes. Since ash is a cohesionless material , most of the settlements occurs during the construction period of embankment Due to lack of sufficient data from BCPP and KSTPP ash dyke sites precise settlement calculations could not be performed. The theoretical results of expected settlement of pond ash below the embankment, is presented here for expected range of site consditions.

Calculations:

The settlement is estimated using compression indexes obtained from laboratory consolidation tests on samples taken KSTPP ash dyke. The settlement calculation of pond ash below Ist and II nd raising during construction period of these ash embankments is summarized in Table 5.4 . Depending on the size of the lagoon, the construction period varies from 3 months to over an year However a conservative value of 3 months is assumed for this calculation Calculation details are given below :

Time of construction of ash embankment = 3 months (assumed)

Compression index $C_c = 0.086$ (Fig 4 20)

$e_o = 0.845$ (Fig. 4.20)

Table 5.4 Calculated Settlements

No. of Raisings	H (m)	σ_o' kg/cm²	S (m)	Single Drainage					Double Drainage				
				T_v	U %	S_t (m)	Sn	% strain	T_v	U %	S_t (m)	Sn	% strain
I	5	975	0 195	0.408	70.34	0.137	0.058	2 74	1 63	98.55	0 192	-	3.84
II	8	1560	0.249	0.195	45.0	0.112	0.137	1 4	0.636	98.77	0.246	-	3 1

Coefficient of Consolidation $C_v = 0.788$ cm²/min (Fig. 4.17)

γ_{sat} of lagoon ash = 1.39 gm/cc (Table 4.3)

γ_{total} of compacted embankment = 1.43 gm/cc (Table 4.3)

The settlement has been calculated using the equation below:

$$S = \frac{C_c H}{1 + e_o} \log_{10} \frac{\sigma_o' + \Delta\sigma_o'}{\sigma_o'}$$

S = total settlement

H = Depth of compressible layer

σ_o' = effective vertical stress at mid depth

under Ist raising

390 kg/cm³ x 2.5m = 975 kg/cm²

under IInd raising

390 kg/cm³ x 4.0 m =1560 kg/cm²

$\Delta\sigma_o'$ = increase in effective vertical stress due to

load of embankment

= 4m x 1.430 kg/cm³ =5720 kg/cm²

Time factor , $$T_v = \frac{C_v t}{d^2}$$

where d = drainage path

$$T_v = 1.781 - 0.933 * \log_{10}(100 - U) \quad \text{for } T_v > 0.2$$

$$T_v = \frac{\pi}{4}(U)^2 \quad \text{for } T_v < 0.2$$

where U = degree of consolidation

S_t = Settlement after time 't'

$= S \times U\%$

Net settlement expected after embankment construction 'S_n' $S_n = S - S_t$

As discussed in section 4.4.2 , the measured settlements are 10 mm and 60 mm at KSTPP and BCPP ash dyke sites respectively. Exact details of both these sites are not available, but it appears that at BALCO ash dyke, the settlement plate was installed prior to construction where as at KSTPP ash dyke, after the construction Since the depth of the settled ash is also not known, therefore no precise calculations could be made about the settlement values

5.4 Lateral Deviation

To measure the lateral displacement of ash dykes inclinometer were installed on the III re raising of KSTPP ash dyke Details of the inclinometer has been discussed in chapter 3 and the data set recorded during investigation has been reported in chapter 4. The results indicate that a maximum movement of 18.6 mm has occurred at a depth of 2 m from the crest of the embankment, towards the downstream side of the dyke The records of incremental displacement exhibits presence of shear zones (Fig 4 10(a) and (b)) at a depth of 2 -3 m and again at 10 m below the crest of embankment which indicates possible slip surface at the interface of IInd and IIIrd raising and at the base of the dyke It is to be noted that these movement has occurred immediately after the commencement of ash slurry discharge in the pond About two months later when the pond was full, the second set of readings were taken and no appreciable displacement could be detected Thus, it could be concluded that the entire ash dyke settles vertically and moves radially outwards only during the filling operation after which negligible movement is expected. However, the movement recorded is insignificant (less than 0.01 %)

5.5 Slope Stability Analysis

5.5.1 Package used - pcstabl

The computer programme used for the analysis of the ash dykes is named as PCSTABL, which is developed to handle general slope stability problems by the simplified Janbu method and the modified Bishop method of slices The Janbu method allows for analysis assuming shear surfaces of general shape. The value of the factor of safety obtained by these limiting equilibrium procedures, although not satisfying complete equilibrium, usually compares favourably to more accurate methods, which are considerably more computing time and require evaluation of results for reasonableness

In this package critical surface searching options include three possible ways in which it generates trial failure surfaces pseudo-randomly of circular shape, sliding block character and general irregular shape Problem capabilities include . general profile definition ; ground surface surcharge boundary loads, pore pressures approximately defined with respect to piezometric surfaces, defined as a constant through a zone, related proportionally to the overburden, or defined as combination of the aforementioned ; and pseudo- static earthquake loading

5.5.2 Factors of Safety

In the stability analysis of slopes, many design factors cannot be determined with certainty . Therefore, a degree of risk should be assessed in an adopted design. The factor of safety fulfills this requirement. The factor takes into account not only the uncertainties in design parameters but also the consequences of failure

A safety factor of at least 1.5 is commonly employed (Lambe and Whitman,1969) for static loading conditions For seismic analysis the recommended values varies between 1 0 and 1.2. Since Indian Standard has not specified any recommendation related to the factor of safety of ash solid waste disposal facilities. Based on the recommendations of other international organizations such as U.S A , U K and Canada the following values of factor of safety is suggested for ash dyke design

F.S. (static) = 1 5 · F.S (dynamic) = 1 1

Since, from field tests on KSTPP ash dyke , an improvement in the strength values of pond ash were found, as compared to the initial strength parameters used during designing of ash dyke a decade ago (refer Table 5 5), therefore, factor of safety for the slope stability of the ash dyke was re-evaluated using improved strength parameters of ash. The slope stability analyses was carried out , assuming the same geotechnical properties of KSTPP ash dyke, for each earth quake zone of India and the results are shown in Table 5 6 A map of India showing seismic zones is shown in Figure 5.1.

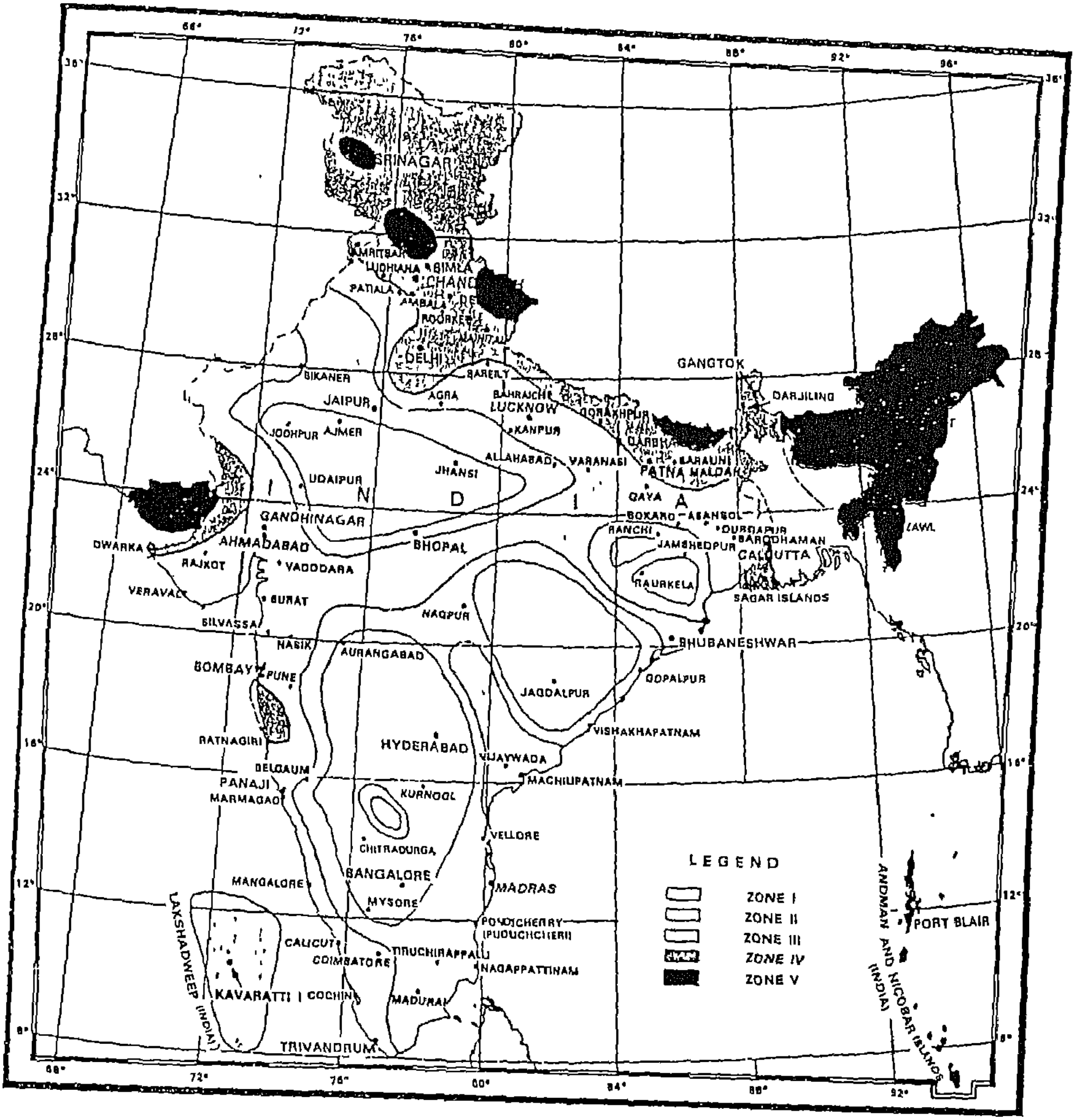
Table 5.5 ϕ values used in stability analyses

Soil Type	$\phi_{initial}$	$\phi_{improved}$
compacted ash (Soil II)	33°	33°
settled ash (Soil III)	28°	31°
foundation soil (Soil 1)	33°	33°
lagoon ash (Soil IV)	28°	28°

Although higher degree of improvement of the lagoon ash were obtained, still the conservative values of ϕ for both compacted and pond ash were used for the stability analysis Factor of Safety as calculated from initial assumed geotechnical design parameter of ash permitted only five raisings at KSTPP ash dyke, which lies in Zone I of India The new IS code is proposing to combine zone I and zone II (code is under preparation), therefore, KSTPP may also be assumed to lie in zone II Using improved strength parameters of ash for stability analysis it is found that eight raisings could be achieved safely. The F.S. of five raisings with initial assumed strength parameters of ash was 1 504 for static loading conditions and 1.04 for seismic loading conditions With improved conditions the F S rises to 1.736 for static loading conditions and 1.236 for dynamic loading conditions Therefore, higher number of raisings are possible at this site. As shown in Table 5 6, eight raisings can be carried out at this site with F.S of 1.52 and a 1 06 for static and dynamic loading conditions respectively. Using the same geotechnical parameters, it has been found that at BTPS site only threee raisings can be carried out because of it being loacated in high seismic zone (zone IV) The controlling F S. remains more than 2. Based on the F S. given in Table 5.6, it is imperative that the upstream method of construction should be avoided in zone V and in zones III and IV construction by upstream method should be restricted to a maximum of two to three raisings.

Table 5.6 : Result of Slope Stability Programme PCSTABL for KSTPP

No. of Raisings	STATIC CONDITIONS		SEISMIC CONDITIONS									
	INITIAL	IMPROVED	INITIAL					IMPROVED				
			Zone I	II	III	IV	V	Zone I	II	III	IV	V
3	1.895	2.142	1.554	1.316	0.994	-	-	1.754	1.484	1.129	1.002	-
4	1.601	1.835	1.335	1.143	0.883	-	-	1.538	1.317	1.015	0.909	-
5	1.504	1.736	1.252	1.067	0.821	-	-	1.450	1.236	0.951	-	-
6	1.428	1.648	1.199	1.028	-	-	-	1.384	1.191	-	-	-
7	1.391	1.546	1.165	0.997	-	-	-	1.301	1.115	-	-	-
8	1.324	1.521						1.286	1.106			



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The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate base line

Responsibility for the correctness of internal details shown on the map rests with the publisher

The boundary of Meghalaya shown on this map is as interpreted from the North-Eastern Areas (Reorganization) Act, 1971, but has yet to be verified

Fig 51 *Map of India Showing Seismic Zones*

Chapter 6

Summary and Conclusion

In this investigation, post construction monitoring of ash dyke raised by upstream method of construction was carried out for two ash disposal sites, Korba Super Thermal Power Plant (KSTPP), Korba and Badarpur Thermal Power Station (BTPS), New Delhi Standard penetration test (SPT), piezocone and plate load tests were performed at BTPS ash dyke site and laboratory tests leading to characterization of geotechnical properties, such as grain size distribution, specific gravity, standard proctor test, permeability, triaxial and direct shear test were also performed for BTPS ash.

Due to logistic problem, piezocone could not be conducted at KSTPP ash dyke site. At this site , only SPT was performed and high precision instruments like inclinometer, for detecting lateral movement was installed in the third raising of the ash dyke For vertical settlements , cross arm settlement plate readings were analyzed and the functioning of filter system was monitored from the piezometer readings Based on the comprehensive field and laboratory investigation programme in-situ strength properties of compacted ash and unconsolidated settled ash were estimated. Slope stability analyses were carried out both for static and dynamic loading conditions to re-evaluate the safety factors for improved strength characteristics of ash through a software 'PCSTABL'.

The following conclusions are drawn based on this investigation :

1) SPT conducted at both KSTPP and BTPS ash dykes have shown marked improvement in the strength of settled ash beneath the embankment as compared to settled ash of other locations in the pond
2) Piezocone tests were also conducted at BTPS ash dyke to investigate improvement in the shear strength of pond ash. This again has demonstrated significant improvement of shear strength in settled ash due to imposition of embankment load over it.
3) q_c / N ratio for ash was found to vary form 5 to 7 which is slightly higher than that of the silty sandy soils
4) The ϕ values near ash-slurry discharge end of BTPS pond is found to be comparatively more than the water outlet end This is attributed to that fact that there is accumulation of finer particles near the outlet end. This can be overcome by discharging the ash in garlanding fashion so that the ash particles are distributed uniformly over the entire pond This will lead to a economical and safe design of ash embankments
5) Shear Strength properties have been calculated from theoretical empirical relationship from the SPT and Piezocone test results The angle of internal friction ϕ of the pond ash is found to improve from 28° to 31°
6) Plate Load test performed on the settled ash have been analyzed and based on test results, the estimated allowable bearing capacity of the settled ash is of the order of 0.4 T/m². The angle of internal friction ϕ has been calculated from the

plate load test results using bearing capacity formulae The values are found to be 30 and 32 which is of the same order as found by other insitu methods

7) Inclinometer was installed at KSTPP ash dyke to monitor the lateral movement of ash embankment Lateral movement was found to occur only during initial phases of ash slurry filling operation in the lagoon after which no appreciable movement was detected The total movement of the Downstream side of embankment of stage IV constructed on the settled ash is less than 0 2 %

8) The piezometers readings and the water level during drilling operations have revealed that the phreatic line is near the drainage blanket. This demonstrates the proper functioning of the internal drainage system installed within the embankment.

9) Based on the laboratory tests it is estimated that for double drainage site conditions, the settlement of lagoon ash is 98 -99 % of total expected settlement during the construction of ash embankment over it Based on actual field measurements, it is estimated that total settlement of the settled ash is likely to be less than 1% during construction of ash embankment and 0.2 % after construction of ash embankment

10) Geotechnical investigations were carried out on samples of BTPS ash as per Indian Standards of Laboratory testing. Some tests which were felt necessary and not performed earlier for KSTPP ash were also conducted. The geotechnical properties of BTPS ash is similar to what has been found for pond ashes of other sites in India

11) The static and dynamic factors of safety increases tremendously on using the improved strength of ash for analysis KSTPP ash dyke was initially designed for five raisings using strength parameters of settled ash without any improvement. With the improved strength properties of the settled ash , it has been found that eight numbers of raisings can be achieved. Based on the insitu and laboratory test results of both sites, maximum embankment raisings has been proposed for various seismic zones in India. As the static factor of safety remains the same for all zones , the controlling factor of safety for the stability

of ash dykes is seismic loading conditions Based on the stability analysis it is conclude that in zones I and II, eight raisings of 3 m each is possible where in zones III and IV, only three raisings are possible. In zone V, raisings should not be done by upstream method of construction, instead down stream method of construction should be used.

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